Engineering Hydrology Lectures – 4th. Stage

Major References :

1. ***Engineering Hydrology by Subramanya***

2. ***Advanced Hydrology by V.T. Chow***

3. ***Engineering Hydrology by Linsley***

Syllabus :

First Semester :

Chapter One : ***Introduction***
Chapter Two : ***Precipitation***
Chapter Three : ***Abstraction from Precipitation***
Chapter Four : ***Run-Off***

Second Semester :

Chapter Five : ***Hydrograph***
Chapter Six : ***Floods***
Chapter Seven : ***Flood Routing***
Chapter Eight : ***Ground Water***

Chapter One

Introduction

1.1. Hydrology :

Hydrology means the science of water. It is the science that deals with the occurrence, circulation and distribution of water of the earth and earth's atmosphere. As a branch of earth science, it is concerned with the water in streams and lakes, rainfall and snow fall, snow and ice on the land and water occuring below earth's surface in the pores of the soil and rocks. In a general sense, hydrology is very broad subject of an inter – disciplinary nature drawing support from allied sciences, such as meteorology, geology, statistics, chemistry, physics and fluid mechanics.

Hydrology is basically an applied science. To further emphasis the degree of applicability, the subject is sometimes classified as :

1. Scientific Hydrology : the study which is concerned chiefly with academic aspects.
2. Engineering or Applied Hydrology : a study concerned with engineering applications.

In a genral sense, engineering hydrology deals with:

a- Estimation of water resources.

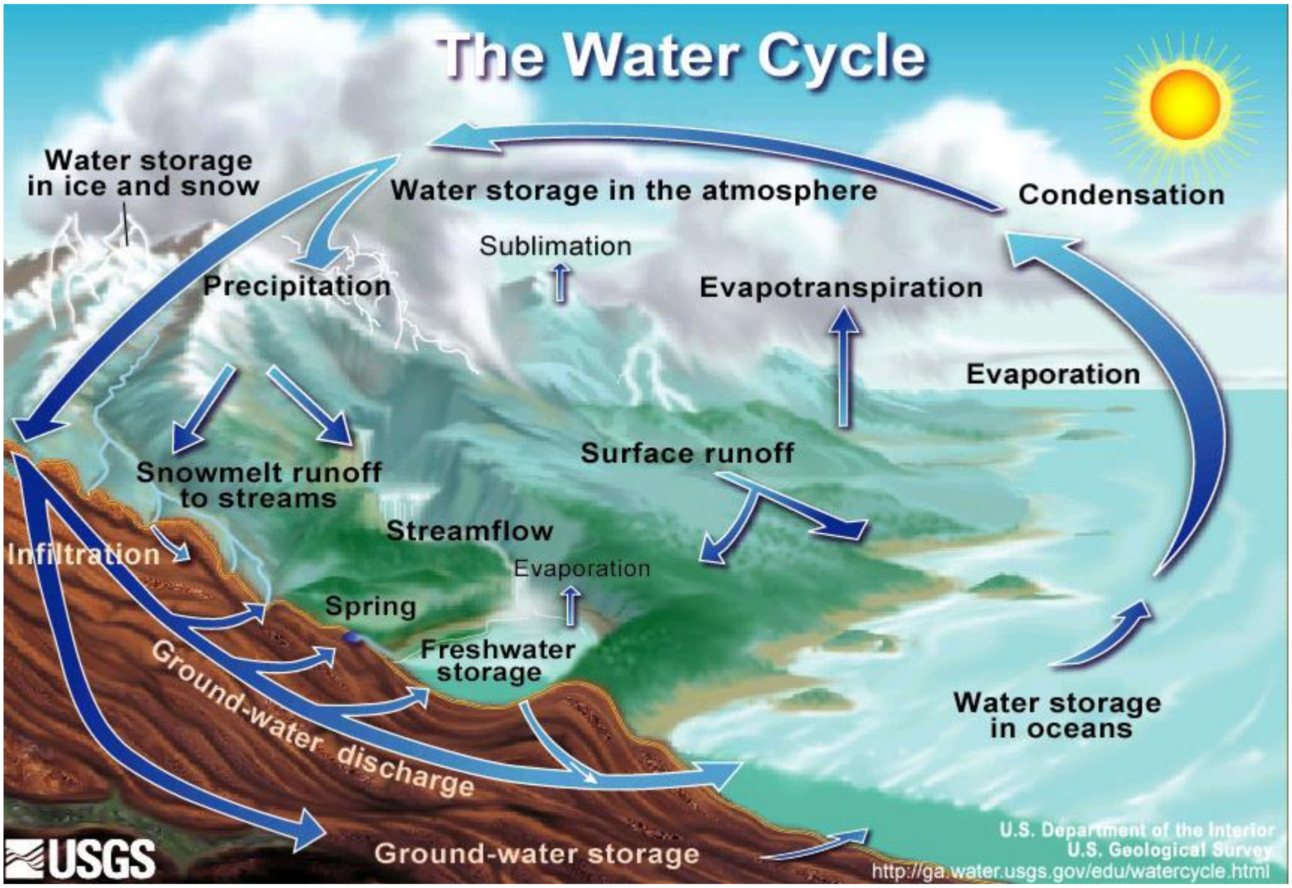
b- The study of processes such as precipitation, runoff, evapotranspiration and their interaction.

c- The study of problems such as floods and droughts, and strategies to combat them.

1.2. Hydrological Cycle :

Water occurs on the earth in all its three states, liquid, solid and gaseous and in various degrees of motion. Evaporation of water from water bodies such as oceans and lakes, formation and movement of clouds, rain and snowfall,

streamflow and ground water movement are some examples of the dynamic aspects of water. The various aspects of water related to the earth can be explained in terms of a cycle known as the *Hydrologic Cycle* as shown in fig. (1).



1.3. Hydrological cycle Paths :

There are large number of paths for water in the hydrologic cycle. Furthermor, it is a continuous recirculating cycle in the sense that there is neither a beginning noe an end or a pause. Each path of the hydrologic cycle involves one or more of the following aspects :

1. Transportation of water.
2. Temporary storage of water.
3. Change of state.

1.4. Water Budget Equation :

For a given problem area, say a catchment, in an interval of time Δt , the continuity equation for water in its various phasis is written as :

$$\boxed{\Delta S = V_i - V_o} \quad \ldots\ldots (1.1)$$

In which :

ΔS : change in the storage of the water volume over and under the given area during the

given period.

V_i : inflow volume of water into a cathment area, and

V_o : outflow volume of water from a catchment area.

Example (1) : A catchment area of 15 km^2 , calculate :

1. **The change in the storage volume (for 1 year) over and under the given catchment if the volume of inflow is $8 * 10^4$ m^3 and for outflow $6.5 * 10^4$ m^3 .**
2. **If the annual average for streamflow is 10^7 m^3 , calculate the equivalent depth.**

Solution :

1. $\Delta S = V_i - V_o$

   $\Delta S = 8 * 10^4 - 6.5 * 10^4 = 1.5 * 10^4 \ m^3$
2. Average Depth = $10^7 / 15*10^6 = 0.667$ m. = 66.7 cm.

While realizing that all the terms in a hydological water budget may not be known to the same degree of accuracy, an expression for the water budget of a catchment for a time interval Δt is written as :

$$\boxed{P - R - G - E - T = \Delta S} \quad \ldots\ldots (2)$$

In this P : Precipitation , R : surface runoff , G : net ground water flow out of the catchment , E : evaporation, T : transpiration and ΔS : change in storage

The storage S consists of three components as :

$$S = S_s + S_m + S_g \quad \text{...... (3)}$$

Where

S_s : surface water storage

S_m : water in storage as soil moisture, and

S_g : water in storage as groundwater.

Thus eq. (3) becomes :

$$\Delta S = \Delta S_s + \Delta S_m + \Delta S_g \quad \text{...... (4)}$$

Example (2) : A lake had a water surface elevation of 103.2 m above datum at the beginning of a certain month. In that month, the lake received an average inflow of 6 m^3/s from surface runoff sources. In the same period, the outflow from the lake had an average value of 6.5 m^3/s. Further, in that month, the lake received a rainfall of 145 mm and the evaporation from the lake surface was estimated as 6.1 cm. Write the water budget equation for the lake and calculate the water surface elevation of the lake at the end of the month. The average lake surface area can be taken as 5000 ha. Assume that there is no contribution to or from the ground water storage.

Solution :

In a time interval Δt , the water budget for the lake can be written as :

$$(\bar{I}\,\Delta t + PA) - (\bar{Q}\,\Delta t + E\,A) = \Delta S$$

Where $\bar{I}$ = average rate of inflow of water into the lake

$\bar{Q}$ = average rate of outflow from the lake

A = average surface area of the lake.

Δt = 1 month = 30 * 24 * 60 * 60 = $2.592*10^6$ s

Inflow volume = $\bar{I}\,\Delta t$ = 6 * 2.592 = 15.552 Mm^3

Outflow volume = $\bar{Q}\,\Delta t$ = 6.5 * 2.592 = 16.848 Mm^3

Input due to precipitation = P A = $\frac{14.5*5000*100*100}{100*10^6}$ = 7.25 Mm^3

Outflow due to evaporation = E A = $\frac{6.1*5000*100*100}{100*10^6}$ = 3.05 Mm^3

Hence ΔS = 15.552 + 7.25 – 16.848 – 3.05 = 2.904 Mm^3

Change in elevation $\Delta z = \frac{\Delta S}{A} = \frac{2.904*10^6}{5000*100*100}$ = 0.058 m

New water surface elevation at the end of the month = 103.2 + 0.058

= 103.258 m above the datum.

Example (3) : A small catchment of area 150 ha received a rainfall of 10.5 cm in 90 minutes due to a storm. At the outlet of the catchment, the stream draining the catchment was dry before the storm and experienced a runoff lasting for 10 hours with an average discharge of 1.5 m^3/s. The stream was again dry after the runoff event. (a) What is the amount of water which was not available to runoff due to combined effect of infiltration, evaporation and transpiration? (b) What is the ratio of runoff to prscipitation?

Solution :

The water budget equation for the catchment in a time Δt is

$$R = P - L$$

Where L : losses (water not available to runoff due to infiltration, evaporation, transpiration and surface storage)

(a) P = Input due to precipitation in 10 hours = 150*100*100*(10.5/100)

= 157500 m^3

R = Runoff volume = outflow volume at the catchment outlet in 10 hours

$R = 1.5*10*60*60 = 54000\ m^3$

Hence Losses $L = 157500 - 54000 = 103500\ m^3$

(b) Runoff / Rainfall = 54000/157500 = 0.343 (Runoff Coefficient)

1.5. Engineering Aplications of Hydrology :

Hydrology finds its greatest application in the design and operation of water resources engineering projects, such as those for :

1. Irrigation
2. Water supply
3. Flood control
4. Water power
5. Navigation

In all projects above, hydrological investigation for the proper assessment of the following factors are necessary :

1. The capacity of storage structures such as rservoirs.
2. The magnitude of flood flows to enable safe disposal of the excess flow.
3. The minimum flow and quantity of flow available at various seasons.
4. The interaction of the flood wave and hydraulic structures, such as levees, reservoirs, barrages and bridges.

1.6. Typical Failure Factors of Hydraulic Structures :

1. Overtopping and consequent failure of an earthen dam due to an inadequate spillway capacity.
2. Failure of bridges and culverts due to excess flood flow.
3. Inability of a large reservoir to fill up with water due to overestimation of the stream flow.

1.7. Sources of Data :

Depending upon the problem at hand, a hydrologist would require data relating to the various relevant phases of the hydrological cycle playing on the problem catchment. The data normally required in the studies are :

1. Weather records – temperature, humidity and wind velocity.
2. Precipitation data.
3. Stream flow records.
4. Evaporation and evapotranspiration data.
5. Infiltration characteristics of the study area.
6. Soils of the area.
7. Land use and land cover.
8. Groundwater characteristics.
9. Physical and geological characteristics of the area.
10. Water quality data.

Chapter Two

Precipitation

1.2. Precipitation: **denotes all forms of water that reach the earth from the atmosphere. The usual forms are rainfall, snowfall, hail, frost and dew. Of all of these, only the first two contribute significant amounts of water.**

For precipitation to form :

1. The atmosphere must have moisture.

2. There must be sufficient nuclei present to aid condensation.

3. Weather conditions must be good for condensation of water vapor to take place.

4. The products of condensation must reach the earth.

2.2. Forms of Precipitation :

The rain is the principal and common form of precipitation. The term rainfall is used to describe precipitation in the form of water drops of sizes larger than 0.5 mm. The maximum size of raindrop is about 6 mm. On the basis of its intensity, rainfall is classified as :

Type	Intensity (mm/hr)
Light Rain	Trace to 2.5 mm/hr.
Moderate Rain	2.5 mmlhr. – 7.5 mm/hr.
Heavy Rain	Over 7.5 mm/hr.

2.3. Adequacy of Rain Gauge Stations :

If there are already some rain gauge stations in a catchment, the optimal number of stations that should exist to have an assigned percentage of error in the estimation of mean rainfall is obtained by statistical analysis as :

$$N = \left(\frac{C_v}{\epsilon}\right)^2 \quad \text{.... (1)}$$

N : Optimal number of stations

∈ : allowable degree of error in the estimation of the rainfall mean

Cv : coefficient of variation of the rainfall values at the existing m stations (in percent)

$$C_v = \frac{100 * \sigma_{m-1}}{\bar{P}} \quad \ldots. (2)$$

$$\sigma_{m-1} = \sqrt{\left[\frac{\sum_1^m (P_i - \bar{P})^2}{m-1}\right]} \quad \ldots. (3)$$

σ_{m-1} : standard deviation

P_i : precipitation magnitude in the i[th] station

$$\bar{P} = \frac{1}{m}\left(\sum_1^m P_i\right) \ldots. (4)$$

<u>Example (1) :</u> A catchment has 6 raingauge stations. In a year, the annual rainfall recorded by the gauges are as follows :

Station	A	B	C	D	E	F
Rainfall (cm)	82.6	102.9	180.3	110.3	98.8	136.7

For a 10% error in the estimation of the mean rainfall, calculate the optimum number of stations in the catchment.

Solution:

m = 6 ; σ_{m-1} = 35.04 ; **€** = 10% $\bar{P}$ = 118.6

C_v = 100 * 35.04 / 118.6 = 29.54

N = 8.7 say 9 stations

Thus, we need 3 additional stations

2.4. Estimation of Missing Data :

Given the annual precipitation values P_1, P_2, P_3, …, P_m at neighboring M stations 1, 2, 3, …, M respectively. It is required to find the missing annual percipitation P_x at a station X not included in the above M stations. Further, the normal annual precipitations N_1, N_2, N_3, …., N_i at each of the above (M+1) stations including station X are known.

2.4.1.Arithmatic Mean Method :

If the normal annual precipitations at various stations are within 10% of the normal annual precipitation at station X, then a simple arithmetic average procedure is followed to estimate P_x, Thus

$$P_x = 1/m\ [P_1+P_2+\ldots\ldots+P_m]\ \ldots.\ (5)$$

m : number of stations

P_x : Missing Precipitation in this period

2.4.2. Normal Ratio Method :

If the normal precipitation vary considerably, then P_x is estimated by weighing the precipitation at the various stations by the ratios of normal annual precipitations. Thus P_x calculated as :

$$P_x = N_x/m\ [P_1/N_1 + P_2/N_2 + \ldots\ldots + P_m/N_m]\ \ldots.\ (6)$$

Example (2) : The normal annual rainfall at stations A, B, C and D in a basin are 80.97, 67.59, 76.28 and 92.01 cm respectively. In the year 1975, the station D was inoperative and the stations A, B and C recorded annual precipitations of 91.11, 72.23 and 79.89 cm respectively. Estimate the rainfall at station D in that year.

Solution :

$$P_D = 92.01/3\ (91.11/80.97 + 72.23/67.59 + 79.89/76.28) = 99.48\ \text{cm.}$$

2.5, Test for Consistency of Records :

If the conditions relevant to the recording of a raingauge station have undergone a significane change during the period of record, inconsistency would araise in the rainfall data of that station. This inconsistency would be felt from the time the significant change took place.

Some of the common causes for inconsistency of record are :

1. Shifting of a raingauge station to a new location.
2. The neighbourhood of the station undergoing a marked change.
3. Change in the ecosystem due to calamities, such as forest fires, land slides.
4. Occurance of observational error from a certain date.

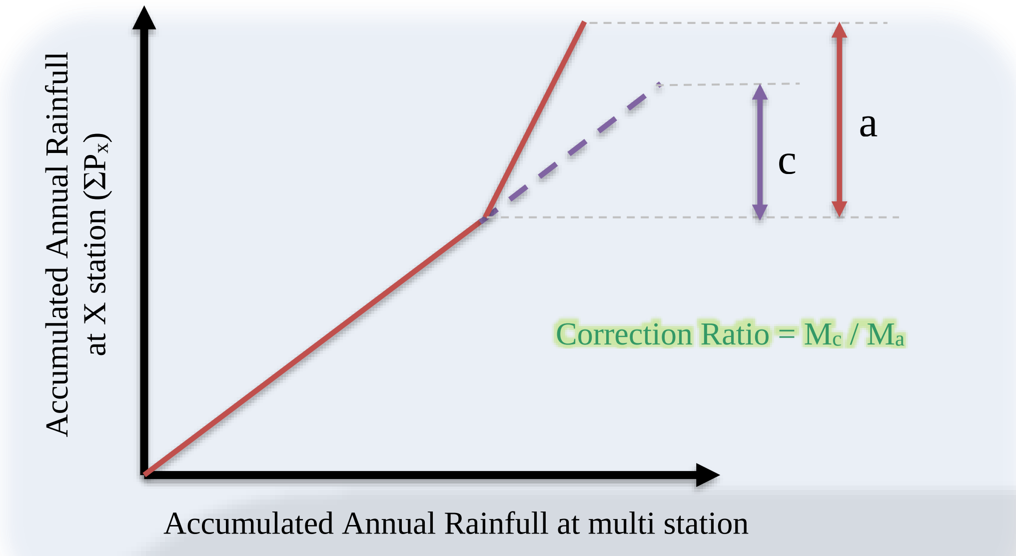
The checking for inconsistency of records is done by the double mass curve technique. This technique is based on the principle that when each recorded data comes from the same parent population, they are consistent.

a. The accumulated precipitation for staton X (i.e. ΣP_x) is calculated. Also the accumulated values for the average rainfall of the group of base stations (i.e. ΣP_{av}) starting from the last record.
b. Plot ΣP_x vs. ΣP_{av}

A decided break in the slope of the resulting plot indicates a change in the precipitation regime of station X. The precipitation values at station X beyond the period of change of regime is corrected by using the relation :

$$P_{cx} = P_x \frac{M_c}{M_a} \quad \ldots. (7)$$

P_{cx} : corrected precipitation at any time period t_1 at station X
P_x : original recorded precipitation at time period t_1 at station X
M_c : corrected slope of double mass curve
M_a : original slope of double mass curve



Example (3) : Annual rainfall Data for station M as well as the average annual rainfll values for a group of ten neighbouring stations located in a meteorologically homogeneous region are given below :

Year	Annual rainfall of station M (mm)	Average Annual Rainfall of the group (mm)	Year	Annual rainfall of station M (mm)	Average Annual Rainfall of the group (mm)
1950	676	780	1965	1244	1400
1951	578	660	1966	999	1140
1952	95	110	1967	573	650
1953	462	520	1968	596	646
1954	472	540	1969	375	350
1955	699	800	1970	635	590
1956	479	540	1971	497	490
1957	431	490	1972	386	400
1958	493	560	1973	438	390
1959	503	575	1974	568	570
1960	415	480	1975	356	377
1961	531	600	1976	685	653
1962	504	580	1977	825	787
1963	828	950	1978	426	410
1964	679	770	1979	612	588

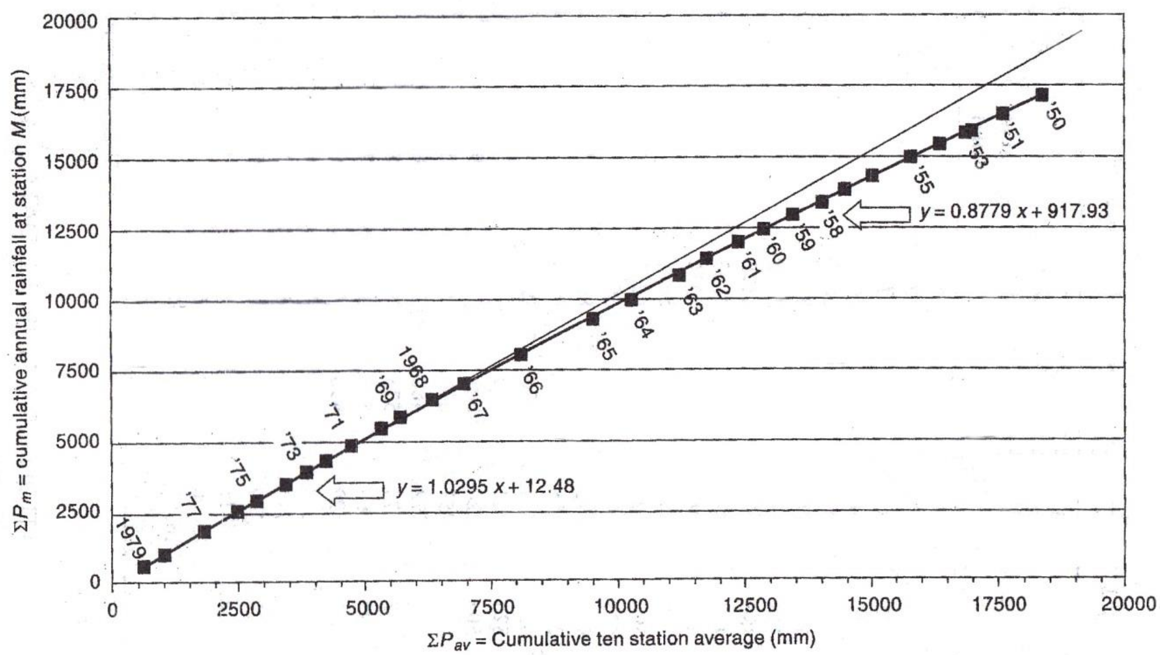
Test the consistency of the annual rainfall data of station M and correct the record if there is any discrepancy. Estimate the mean annual precipitation at station M.

Solution :

Year	P_m (mm)	ΣP_m (mm)	P_{av} (mm)	ΣP_{av} (mm)	Adjusted values of P_m (mm)	Finalised values of P_m (mm)
1979	612	612	588	588		612
1978	426	1038	410	998		426
1977	825	1863	787	1785		825
1976	685	2548	653	2438		685
1975	356	2904	377	2815		356
1974	568	3472	570	3385		568
1973	438	3910	390	3775		438
1972	386	4296	400	4175		386
1971	497	4793	490	4665		497
1970	635	5428	590	5255		635
1969	375	5803	350	5605		375
1968	596	6399	646	6251	698.92	699
1967	573	6972	650	6901	971.95	672
1966	999	7971	1140	8041	1171.51	1172
1965	1244	9215	1400	9441	1458.82	1459
1964	679	9894	770	10211	796.25	796
1963	828	10722	950	11161	970.98	971
1962	504	11226	5801	11741	591.03	591
1961	531	11757	600	12341	622.7	623
1960	415	12172	480	12821	486.66	487
1959	503	12675	575	13396	589.86	590
1958	493	13168	560	13956	578.13	578
1957	431	13599	490	14446	505.43	505
1956	479	14078	540	14986	561.72	562
1955	699	14777	800	15786	819.71	820
1954	472	15249	540	16326	553.51	554
1953	462	15711	520	16846	541.78	542
1952	95	15806	110	16956	111.41	111
1951	578	16384	660	17616	677.81	678
1950	676	17060	780	18396	792.73	793

Total of P_m = 19004 mm
Mean of P_m = 633.5 mm

The data is sorted in descending order of the year, starting from the latest year 1979. Cumulative values of station M rainfall (ΣPm) and the 10 stations average rainfall



Double Mass Curve of Annual Rainfall at Station *M*

values (ΣPav) are calculated as shown in the previous table. The data is then plotted as below :

It is seen that the data plots as two straight lines with a break of grade at the year 1968.

The slope of the best straight line for the period 1979 – 1969 is :

Mc = 1.0295

The slope of the best straight line for the period 1968 – 1950 is :

Ma = 0.8779

Thus, the correction ratio is :

$$\frac{M_c}{M_a} = \frac{1.0295}{0.8779} = 1.173$$

The adjusted values at station M are shown in column 5 of the previous table.the finalized values of P_m for all 30 years of records are shown in column 7.

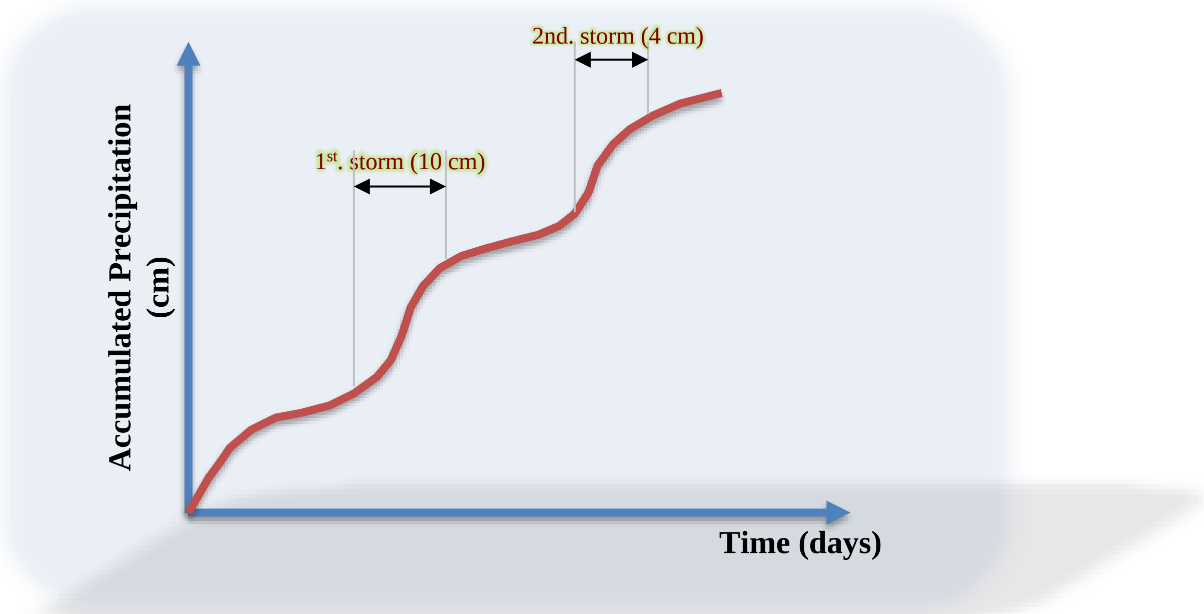
2.6. Presentation of Rainfall Data :

A few commonly used methods of presentation of rainfall data which have been found to be useful in interpolation and analysis of such data are given as follows :

2.6.1. Mass Curve of Rainfall Data :

Is a plot of the accumulated precipitation against time (as shown in figure below). Mass curve is useful in :

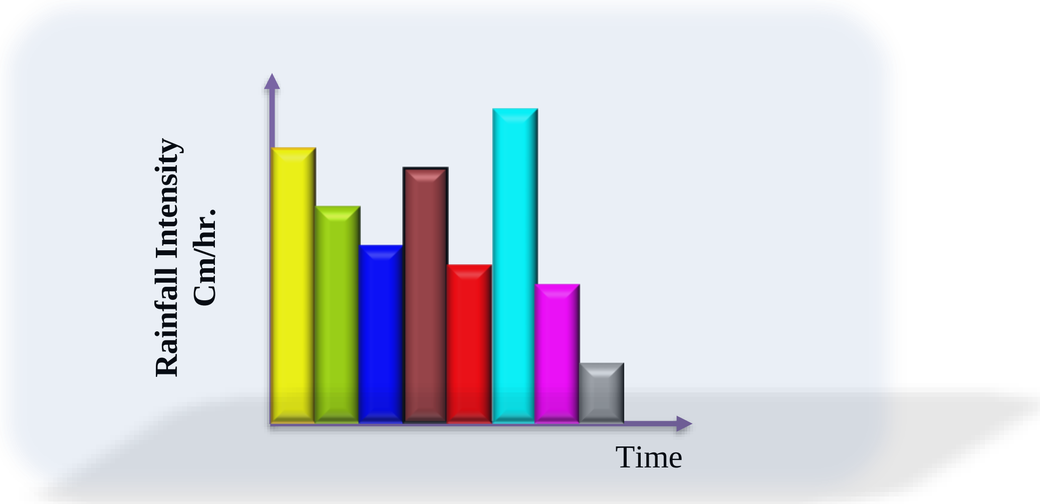
1. Extracting the information on the duration and magnitude of a storm.
2. Intensities at various time intervals in a storm can be obtained by the slope of the curve.



2.6.2. Hyetograph :

Is a plot of the intensity of rainfall against the time interval. The hyetograph is derived from the mass curve and is usually represented as a bar chart (as shown in the next figure). It is very convenient way of :

1. Representing the characteristics of a storm.
2. The development of design storms to predict extreme floods.
3. The area under hyetograph represents the total precipitation received in the period.



2.6.3. Point Rainfall :

Also known as station rainfall refers to the rainfall data of a station. Depending upon the need, data can be listed as daily, weekly, monthly or annual values for various periods. Graphically, these data are represented as plots of magnitude vs. chronological time in the form of a bar diagram. The trend of plot is often discerned by the method of moving average (Moving Mean).

2.6.3.1. Moving Mean Method :

Is a technique for smoothening out the high frequency fluctuations of time series and to enable the trend, if any, to be noticed. The basic principles is that a window of time range m years is selected. Starting from the first set of m years of data, the average of the data of m years is calculated and placed in the middle year of the range m. The window is next moved sequentially one time unit (year) at a time and the mean of the m terms in the window is determined at each window location. The value of m can be 3 or more years (usually an odd value).

Example (4) : Annual rainfall values recorded at station M for the period 1950 to 1979 (given in the previous example). Represent this data as a bar diagram with time in chronological order.

i) Identify those years in which the annual rainfall is :

a) Less than 20 % of mean

b) More than the mean

ii) Plot the three –year moving mean of the annual rainfall time series

Solution :

Figure below shows the bar chart with height of the column representing the annual rainfall depth and the position of the column representing the year of occurrence. The time is arranged in chronological order.

1	2	3	4
Year	Annual rainfall (mm) P_i	3 cumulative year Total for moving mean ($P_{i-1} + P_i + P_{i+1}$)	3- year moving mean (col. 3/3)
1950	676		
1951	578	676+578+95 = 1349	449.7
1952	95	578+95+462 = 1135	378.3
1953	462	95+462+472 = 1029	343.0
1954	472	462+472+699 = 1633	544.3
1955	699	472+699+479 = 1650	550.0
1956	479	699+479+431 = 1609	536.3
1957	431	479+431+493 = 1403	467.7
1958	493	431+493+503 = 1427	475.7
1959	503	493+503+415 = 1411	470.3
1960	415	503+415+531 = 1449	483.0
1961	531	415+531+504 = 1450	483.3
1962	504	531+504+828 = 1863	621.0
1963	828	504+828+679 = 2011	670.3
1964	679	828+679+1244 = 2751	917.0
1965	1244	679+1244+999 = 2922	974.0
1966	999	1244+999+573 = 2816	938.7
1967	573	999+573+596 = 2168	722.7
1968	596	573+596+375 = 1544	514.7
1969	375	596+375+635 = 1606	535.3
1970	635	375+635+497 = 1507	502.3
1971	497	635+497+386 = 1518	506.0
1972	386	497+386+438 = 1321	440.3
1973	438	386+438+568 = 1392	464.0
1974	568	438+568+356 = 1362	454.0
1975	356	568+356+685 = 1609	536.3
1976	685	356+685+825 = 1866	622.0
1977	825	685+825+426 = 1936	645.3
1978	426	825+426+612 = 1863	621.0
1979	612		

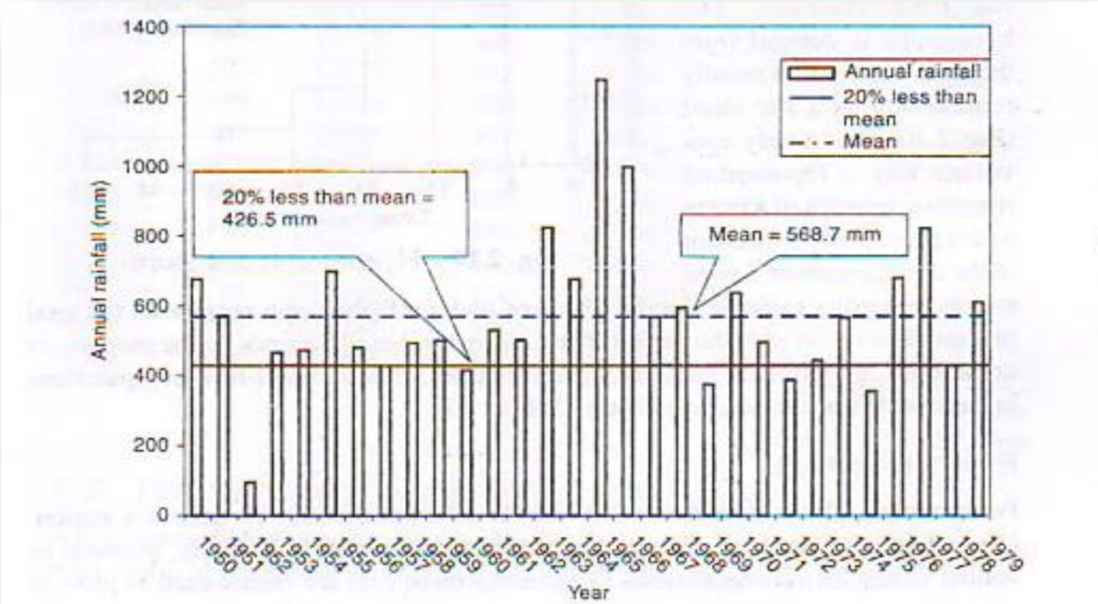


Fig. 2.11 Bar Chart of Annual Rainfall at Station *M*

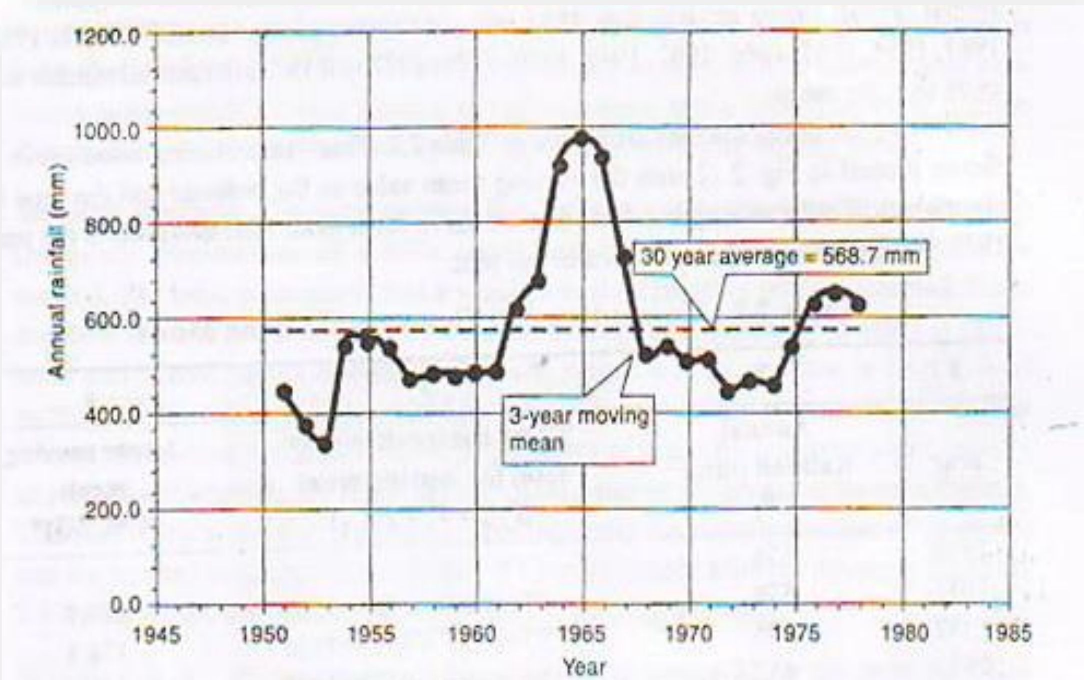


Fig. 2.12 Three-year Moving Mean

2.7. Mean Precipitation Over An Area :

Mean precipitation over an area can be calculated using the following methods :

2.7.1 Arithmatical Mean Method :

$$\bar{P} = \frac{P_1+P_2+\cdots+P_i+\cdots+P_n}{N} = \frac{1}{N}\sum_{i=1}^{N} P_i \quad \ldots.(8)$$

Where P_1 , P_2 , … , P_i , …, P_n are the rainfall values in a given period in N stations within a catchment.

2.7.2. Thiessen Average Method :

$$\bar{P} = \frac{P_1A_1+P_2A_2+\cdots+P_iA_i+\cdots+P_mA_m}{(A_1+A_2+\cdots+A_i+\cdots+A_m)} = \frac{1}{A}\sum_{i=1}^{M} P_iA_i = \sum_{i=1}^{M} P_i\frac{A_i}{A} \quad \ldots.(9)$$

Where P_1 , P_2 , … , P_i , …, P_n are the rainfall values recorded by the stations 1 , 2 , …, i , … , m respectively

A_1 , A_2 , …, A_i , … , A_m are the respective areas of the Thiessen polygons.

2.7.3. Isohyetal Method :

An Isohyet is a line joining points of equal rainfall magnitude.

$$\bar{P} = \frac{a_1\left(\frac{P_1+P_2}{2}\right)+a_2\left(\frac{P_2+P_3}{2}\right)+\ldots\ldots+a_{n-1}\left(\frac{P_{n-1}+P_n}{2}\right)}{A} \quad \ldots.(10)$$

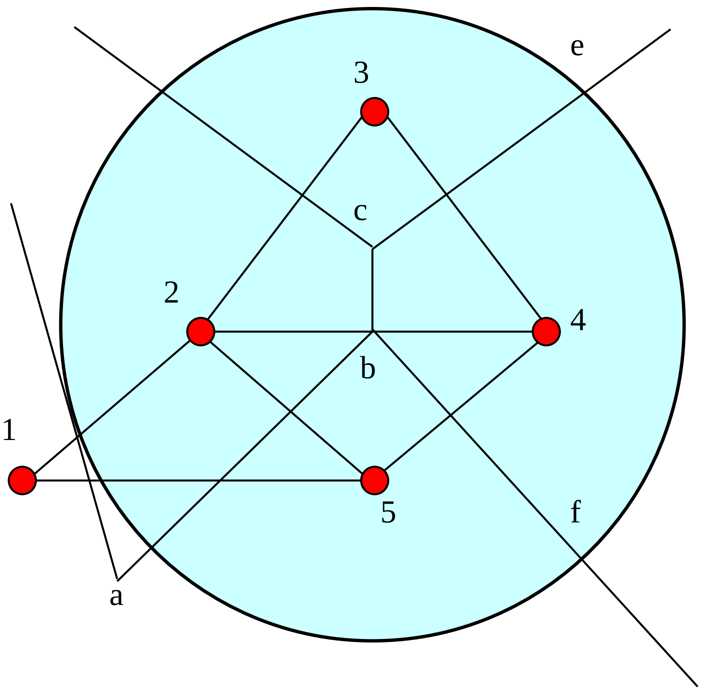
P_1 , P_2 , …., P_{n-1} , P_n : the values of isohyets

a_1 , a_2 , …., a_{n-1} , a_n : the inter isohyet areas

Example (5) : In a catchment area, approximated by a circle of diameter 100 km. Four rainfall stations are situated inside the catchment and one station is outside in its neighbourhood. The coordinates of the center of the catchment and of the five stations in 1980. Determine the average annual precipitation by the Thiessen – mean method.

Station	Center	1	2	3	4	5
Coordinates (km)	(100,100)	(30,80)	(70,100)	(100,140)	(130,100)	(100,70)
Precipitation (cm)	----	85.0	135.2	95.3	146.4	102.2

Solution :

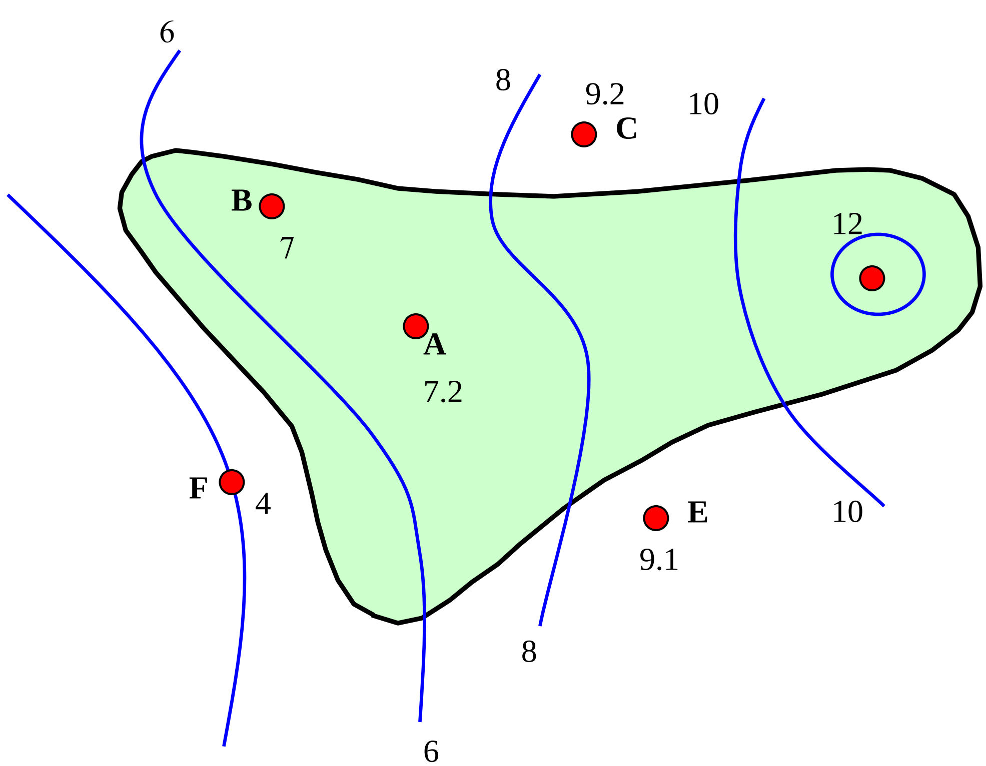


Station	Boundary of Area	Area (Km^2)	Fraction of Total Area	Rainfall	Weighted P (cm)
1	–	–	–	85	–
2	Abcd	2141	0.2726	135.2	36.86
3	Dce	1609	0.2049	95.3	19.53
4	Ecbf	2141	0.2726	146.4	39.91
5	fba	1963	0.2499	102.2	25.54
Total		7854	1.000		121.84

Mean Precipitation = 121.84 cm

Example (6) : The isohyets due to a storm in a catcment were drawn in figure below, and the area of the catchment bounded by isohyets were tabulated as below :

Isohyets	Area (km^2)
12	30
12 – 10	140
10 – 8	80
8 – 6	180
6 - 4	20



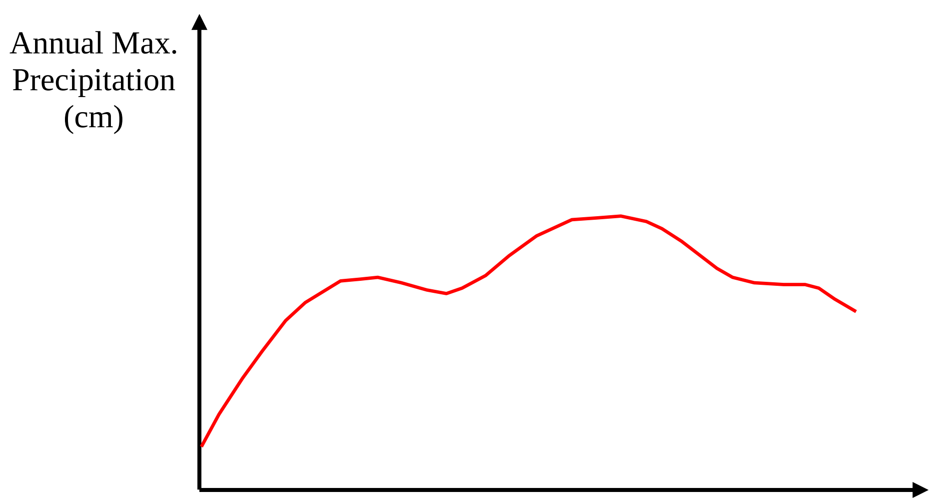
Solution :

Isohyets	Average value of P (cm)	Area (km^2)	Fraction of total area	Weighted P (cm)
12	12	30	0.0667	0.800
12 – 10	11	140	0.3111	3.422
10 – 8	9	80	0.1778	1.600
8 – 6	7	180	0.4000	2.800
6 - 4	5	20	0.0444	0.222
Total		450	1.0000	8.844

Mean Precipitation = 8.844 cm

2.8. Frequency of Point Rainfall :

In many hydraulic engineering applications such as those concerned with floods, the probability of occurrence of a particular extreme rainfall. Such information is obtained by the frequency analysis of the point-rainfall data.



If the probability of an event occuring is (P) its magnitude is equal to or in excess of a specified magnitude X. The return period T is defined as:

$$T = 1/P \quad \text{.......................(11)}$$

Thus, if it is stated that the return period of rainfall of 20 cm in 24 hour is 10 years at a certain station A, it implies that on an average rainfall magnitudes equal to or greater than 20 cm in 10 years, i.e. in a long period of say 100 years, 10 such events can be expected. However, it does not mean that every 10 years one such event is likely, i.e. periodicity is not implied. The probability of a rainfall of 20 cm in 24 hour occuring in anyone year at station A is :

$$P = 1/T \quad \text{..................... (12)}$$

The probability of the event (not occuring) in a given year is (q = 1 – P)

$$q = 1 - P \quad \text{..................... (13)}$$

the probability of the event r times in n successive years is :

$$P_{r,n} = \frac{n!}{(n-r)!r!} P^r q^{n-r} \quad \text{............. (14)}$$

For example :

a. **The probability of an event of exceedence probability P occuring 2 times in n successive years is :**

$$P_{2,n} = \frac{n!}{(n-2)!2!} P^2 q^{n-2} \quad \text{............. (14-a)}$$

b. **The probability of an event not occuring at all in n successive years is:**

$$P_{0,n} = q^n = (1-P)^n \quad \text{...............(14-b)}$$

c. **The probability of an event occuring at least once in n successive years :**

$$P_1 = 1-q^n = 1-(1-P)^n \quad \text{.............(14-c)}$$

Example (7) : Analysis of data on maximum one-day rainfall depth at a specified region that a depth of 280 mm had a return period of 50 years. Determine the probability of a one – day rainfall depth equal to or greater than 280 mm at this region (a) once in 20 successive years , (b) two times in 15 successive years, and (c) at least once in 20 successive years.

Solution :

a) n = 20 , r = 1 , T = 50 , P = 1/50 = 0.02

P1,20 = (20 !)/(19! * 1!) * 0.02 * (0.98)19 = 0.272

b) n = 15 , r = 2

P2,15 = (15!)/(13!*2!)*(0,02)2 * (0.98)13 = 0.0323

c) P1 = 1- (0.98)20 = 0.332

2.9. Plotting Position Criterea :

The purpose of the frequency analysis of an annual series is to obtain a relation between the magnitude of the event and its probability of exceedence. The probability analysis may be made either by empirical or by analytical methods. A simple empirical technique is to arrange the given annual extreme series in descending order of magnitude and to assign an order number m. Thus for the first entry m = 1, for the second entry m = 2 and so on , till the last event for which m = N = number of years of records.

The probability P of an event equalled to or exceeded is given by the *Weibull formula* :

$$P = \left(\frac{m}{N+1}\right) \text{ ..... } (15)$$

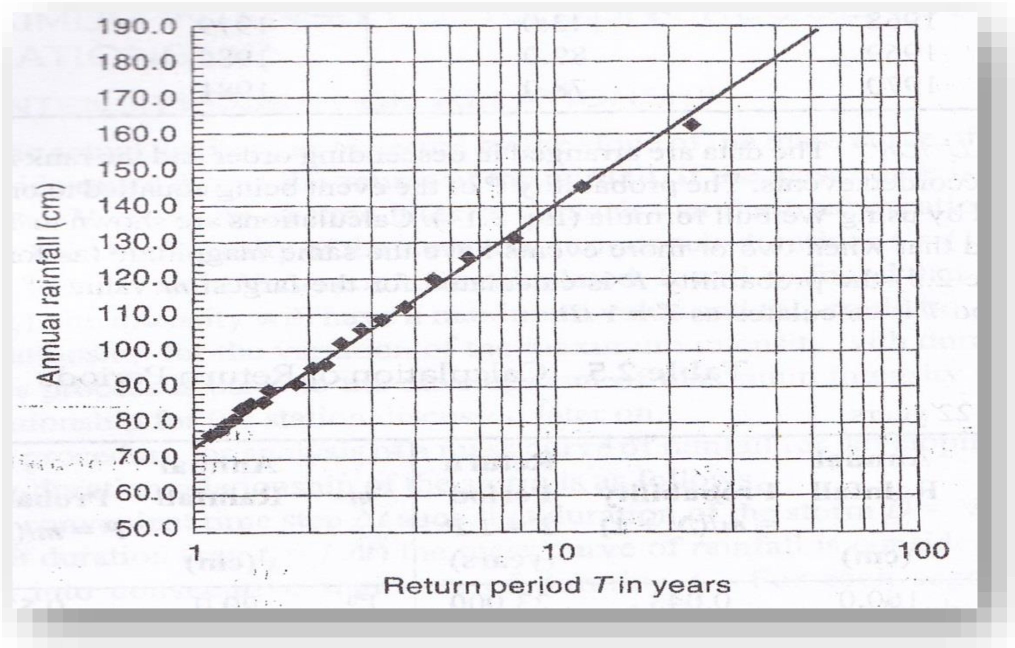
Example (8) : The record of annual rainfall at station A covering a period of 22 years is given below :

Year	60	61	62	63	64	65	66	67	68	69	70
Rainfall (cm)	130	84	76	89	112	96	80	125	143	89	78
Year	71	72	73	74	75	76	77	78	79	80	81
Rainfall (cm)	90	102	108	60	75	120	160	85	106	83	95

a) Estimate the annual rainfall with return periods of 10 years and 50 years.
b) What would be the probability of an annual rainfall of magnitude equal to or exceeding 100 cm occuring at station A?
c) What is the 75% dependable annual rainfall at station A?

M	Rainfall (cm)	P= m/(N+1)	T=1/P	m	Rainfall (cm)	P= m/(N+1)	T=1/P
1	160	0.043	23.26	12	90	0.522	1.92
2	143	0.087	11.5	13	89	-	-
3	130	0.13	7.67	14	89	0.609	1.64
4	125	0.174	5.75	15	85	0.652	1.53
5	120	0.217	4.6	16	84	0.696	1.44
6	112	0.261	3.83	17	83	0.739	1.35
7	108	0.304	3.29	18	80	0.783	1.28
8	106	0.348	2.88	19	78	0.826	1.21
9	102	0.391	2.56	20	76	0.87	1.15
10	96	0.435	2.3	21	75	0.913	1.1
11	95	0.478	2.09	22	60	0.957	1.05

Solution:



a)

T (year)	Rainfall (cm)
10	137.9
50	180

b) Rainfall = 100 cm , thus from the graph T = 2.4 year , then P = 0.417

c) P = 0.75 , T = 1/0.75 = 1.33 year , then Rainfall = 82.3 cm.

Chapter Three

Abstraction from Precipitation

3.1. Evaporation : is the process in which a liquid changes to the gaseous state at the free surface, below the boiling point through the transfer of heat energy.

The rate of evaporation is depenent on :

1. The vapor pressure at the water surface and air above
2. Air and water temperature
3. Wind speed
4. Atmospheric pressure
5. Quality of water
6. Size of the water body

3.1.1. Vapour pressure :

The rate of evaporation is prportional to the difference between the saturation vapour at the water temperature, e_w and the actual vapour pressure in the air, e_a, thus :

$$EL = C\,(e_w - e_a) \;\ldots\ldots\; (1)$$

E_L : rate of evaporation (mm/day)

C : constant

The above equation is known as (Dalton's law of evaporation , 1802)

Note that evaporation continues till $e_w = e_a$ and if ($e_w > e_a$), condensation will takeplace.

3.1.2. Temperature :

Other factors remaining the same, the rate of evaporation increases with an increase in the water temperature.

3.1.3. Wind Speed :

Wind aids in removing the evaporated water vapour from the zone of evaporation and consequently creates greater scope for evaporation. However, if the wind velocity is large enough to remove all the evaporated water vapour, any further increase in wind velocity does not influence the evaporation. Thus the rate of evaporation increases with the wind speed up to a critical speed beyond which any further increase in the wind speed has no influence on the evaporation rate.

3.1.4. Atmospheric Pressure :

Other factors remaining same, a decrease in the barometric pressure, as in high altitudes, increases evaporation.

3.1.5. Soluble Salts :

When a soluble salts is dissolved in water, the vapor pressure of the solution is less than that of pure water and hence causes reduction in the rate of evaporation.

3.2. Evaporimeter :

The amount of water evaporated from a water surface is estimated by the following methods :

1. **Using evaporation data**
2. **Empirical evaporation equations**
3. **Analytical methods**

3.3. Evaporation Stations :

The WMO recommends the minimum network of evaporimeter stations as below :

1. **Arid zones : One station for every 30000 km^2**
2. **Humid temperature climates : One station for every 50000 km^2**
3. **Cold region : one station for every 100000 km^2.**

3.4. Empirical Evaporation Equations :

A large number of empirical equations are available to estimate lake evaporation using commonly available meteorological data. Most formulae are based on the Dalton- type equation and can be expressed as :

$$EL = K\, f(u)\, (e_w - e_a) \quad ..... (2)$$

K : a coefficient

f(u) : wind speed correction function

3.4.1. Meyer's Equation :

$$E_L = k_m\, (e_w - e_a)\, (1 + \frac{U_9}{16}) \quad ..... (3)$$

U_9 : monthly mean wind velocity (km/hr) at about 9 m above ground.

K_m : coefficient accounting for various other factors with a value of 0.36 for large, deep water and 0.5 for small, shallow waters.

3.4.2. Rohwer's Equation :

$$E_L = 0.771\, (1.465 - 0.000732\, P_a)\, (0.44 + 0.0733\, V_o)\, (e_w - e_a) \quad .... (4)$$

Pa : mean barometric reading in mmHg

Vo : mean wind velocity in km/hr at ground level, which can be taken to be the velocity at 0.6 m height above ground

Note :

1. **e_w is found from table (3-3) page No. 72**
2. **Wind velocity at any height above ground (U_h) by knowing any wind speed (U) according to the following equation :**

$$U_h = U\, (\, h\,)^{1/7} \quad ... (5)$$

<u>Example (1) :</u> a) A reservoir with a surface area of 250 hectares had the following average values of climate parameters during a week :

Water temperature 20^o C , relative humidity = 40% , wind velocity at 1 m above ground surface = 16 km/hr. Estimate the average daily evaporation from the lake using Meyer's formula

b) If the evaporation from a pan is indicated as 72 mm in a week :

i) Estimate the accuracy if Meyer's method relative to the pan evaporation measurements.

ii) Eastimate the volume of water evaporated from the lake in that week.

<u>Solution :</u>

a) From table (3-3) , e_w = 17.54 mmHg

e_a = 0.4 * 17.54 = 7.02 mmHg

$U_9 = U_1 * (9)^{1/7} = 16 * (9)^{1/7} = 21.9$ km/hr

Using Meyer equation :

E_L = 0.36 (17.54 – 7.02) (1 + 21.9/16) = 8.97 mm/day

b)

i) Daily evaporation as per pan evaporimeter = (72/7)*0.8 = 8.23 mm/day

Error = (8.23 – 8.97) = - 0.74 mm (Meyer's formula overestimates th evaporation relative to the pan)

Percentage over estimation by Meyer's formula = (0.74/8.23)*100 = 9%

ii) Then evaporated water volume in 7 days (m^3) is :

$V = 7 * (8.23/1000) * 250 * 10^4 = 144025\ m^3$

3.5. Analytical methods of Evaporation Estimation :

The analytical methods for the determination of lake evaporation can be broadly classified into three categories as :

1. **Water Budget Method**
2. **Energy Balance Method**
3. **Mass Transfer Method**

3.5.1. Water Budget Method :

$$P + V_{ig} + V_{is} = V_{og} + V_{os} + E_L + \Delta S + T_L$$

Or : $$E_L = P + (V_{is} - V_{so}) + (V_{ig} - V_{og}) - T_L - \Delta S \quad \ldots\ldots (6)$$

P : daily precipitation

V_{ig} : daily ground water inflow

V_{og} : daily ground water outflow (seepage)

V_{is} : daily surface inflow into the lake

E_L : daily lake evaporation

V_{os} : daily surface outflow from the lake

T_L : daily transpiration loss

ΔS : increase in lake storage in a day

3.6. Evapotranspiration Equations :

3.6.1. Penman's Equation :

$$PET = \frac{A\,H_n + E_a Y}{A + Y} \quad \ldots\ldots (7)$$

PET : daily potential evapotranspiration (mm/day)

A : slope of the saturation vapor pressure vs. temperature curve at the mean air temperature (mmHg/C°) (table 3.3) page [72]

H_n : net radiation in mm of evaporable water per day

E_a : parameter including wind velocity and saturation deficit

Y : constant equal to 0.49 mmHg /C°

The net radiation (H_n) is estimated by the following formula :

$$H_n = Ha(1- r)\,(a+b(n/N)) - \sigma Ta^4\,(0.56 - 0.092\sqrt{e_a})\,(0.1 + 0.9\,(n/N)) \quad \text{.... (7.1)}$$

Ha : incident solar radiation outside the atmosphere on a horizontal surface, expressed in mm of evaporable water per day (it is a function of latitude as indicated in table 3.4 Page[72])

$$a = 0.29 \cos \Phi \quad \text{..... (7.2)}$$

b = 0.52

Φ : North Latitude

n : actual duration of bright sunshine (hrs.)

N : maximum possible hours of bright sunshine (it is a function of latitude as indicated in table 3.5 Page[73])

r : reflection coefficient

σ : Stefan - Boltzman constant = $2.01 * 10^{-9}$ mm/day

T_a = 273 + C°

e_a : actual mean vapor pressure in the air (mmHg)

$$Ea = 0.35\,(1 + (U_2 /160))\,(e_w - e_a) \quad \text{..... (7.3)}$$

U_2 : mean wind speed at 2 m above ground in (km/day)

Example (2) : Calculate the potential evapotranspiration from an area in the month of November by Penman's formula. The following data are available :

Latitude = 28° 4' N

Mean monthly temperature = 19° C

Mean relative humidity = 75 %

Wind velocity at 2 m height = 85 km/day

r = 0.25

Solution :

From table 3.3 :

A = 1 mmHg /Co , ew = 16.5 mmHg

From table 3.4 :

Ha = 9.506 mm of water / day

From table 3.5 :

N = 10.716 hr.

n / N = 9 / 10.716 = 0.84

ea = 0.75 * 16.5 = 12.38 mmHg

a = 0.29 cos 28° 4' = 0.2559 , b = 0.52 , $\sigma = 2*10^{-9}$

Ta = 273 + 19 = 292 k , $\sigma Ta^4 = 14.613$, r = 0.25

Hn = 9.506(1 - 0.25) (0.2559 + 0.52*0.84) – 14.613(0.56 – 0.092)(0.1 + 0.9 (0.84))

Hn = 1.99

Ea = 0.35 (1 + (85/100)) (16.5 – 12.38) = 2.208 , Y = 0.49

$$\text{PET} = \frac{(1*1.99)+(2.208*0.49)}{1+0.49} = 2.06 \text{ mm/day}$$

3.6.2. Blaney – Criddle Formula :

$$\text{PET} = 2.54\ K\ F \ldots\ldots. (8)$$

$$F = \Sigma\, P_h\, \overline{T_f} / 100 \ldots\ldots (8.1)$$

PET : potential evapotranspiration (cm)

K : an empirical coefficient depending on the type of the crop and stage of growth

F : sum of monthly consumptive use factors for the period

P_h : monthly percent of annual day – time hours, depends on the latitude of the place (table 3.6 Page[75])

$\overline{T_f}$: mean monthly temperature in F°

Example (3) : Estimate the PET of an area for the season of November to February in which wheat is grown. The area is at 30° N with mean monthly temperature as below (use k = 0.65) :

Month	November	December	January	February
Temperature (C°)	16.5	13	11	14.5

Solution :

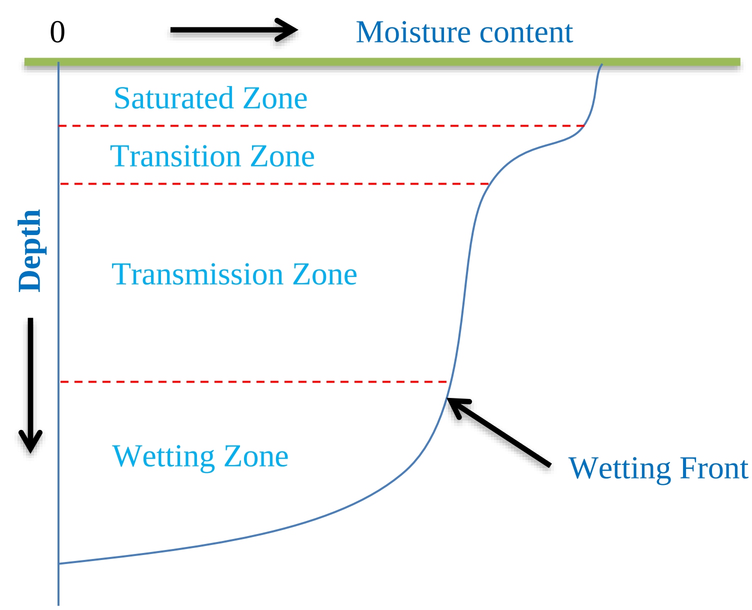
Month	T_f (F°)	P_h	$P_h\ T_f$ / 100
November	61.7	7.19	4.44
December	55.4	7.15	3.96
January	51.8	7.3	3.78
February	58.1	7.03	4.08

Total = 16.26

PET = 2.54 * 0.65 * 16.26 = 26.85 cm.

3.7. Infiltration :

Is the flow of water into the ground through the soil surface. The distribution of soil moisture within the soil profile during the infiltration process is illustrated in figure below :



When water is applied at the surface of a soil, four moisture zones in the soil, as indicated in the figure below can be identified :

1. **Zone 1 : at the top, a thin layer of saturated zone is created.**
2. **Zone 2 : beneath zone 1, there is a transition zone.**

3. **Zone 3 : Next lower zone is the transmission zone where the downward motion of the moisture takesplace. The moisture content in this zone is above field capacity but below saturation.**
4. **Zone 4 : it is called wetting zone and the soil moisture in this zone will be at or near field capacity and the moisture content decreases with the depth.**

3.8. Depression Storage:

When the precipitation of a storm reaches the ground, it must first fill upall depressions before it can flow over the surface. The volume of water trapped in these depressions is called depression storage.

Depression storage depends on :

1. **The type of soil**
2. **The condition of the surface reflecting the amount and nature of depression**
3. **The slope of the catchment**
4. **The antecedent precipitation, as a measure of the soil moisture.**

3.9. Infiltration Capacity :

The maximum rate at which a given soil at a given time can absorb water. It is designatedc as f_p (cm/hr.)

The actual rate of infiltration f_p can be expressed as :

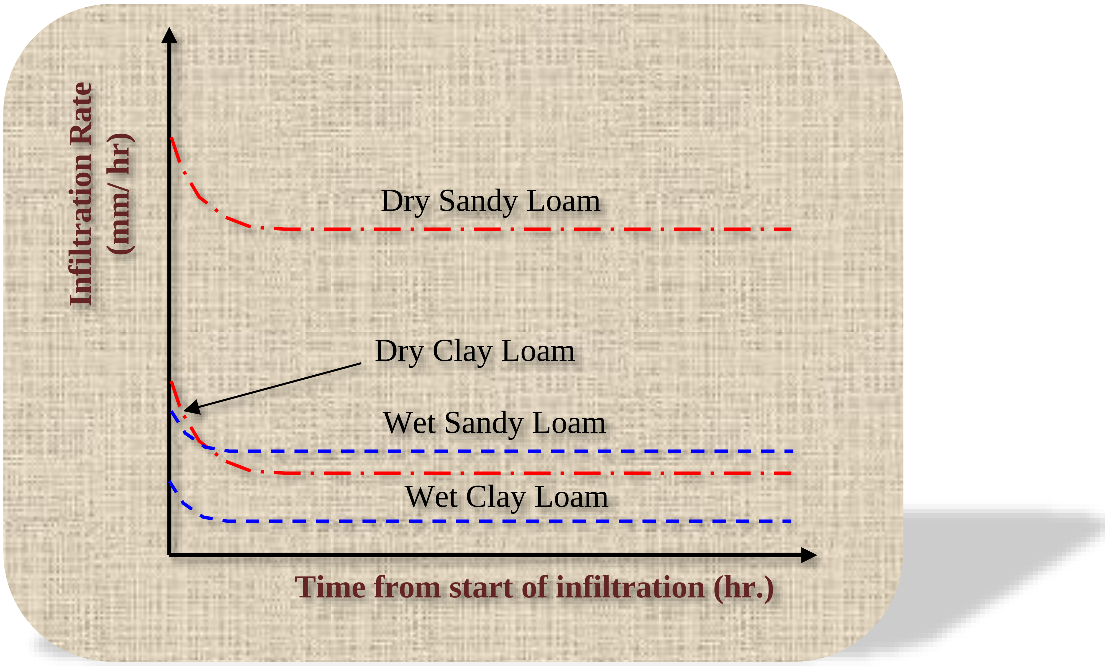
$f = f_p$ if $i \geq f_p$

$f = i$ if $i < f_p$

i : rainfall intensity

The infiltration capacity of an area is depending on :

1. **Characteristics of soil (texture, porosity and hydraulic conductivity)**



Referring to the figure above, there are two types of soils (sandy loam and clay loam) at different initial conditions (dry or wet). The figure shows the optimum variation at infiltration capacity for the soils mentioned above which is high at the beginning of a storm and has an exponential decay as the time elapses depending on Horton equation (Horton representation, 1930) :

$$fc_t = fc_f + (fc_o - fc_f)\, e^{-k_h t} \quad \ldots\ldots (9) \qquad\qquad 0 \leq t \leq t_d$$

fc_t : Infiltration capacity at any time from the beginning of rainfall

fc_o : Initial infiltration capacity at t = 0

fc_f : Terminal infiltration capacity at $t = t_d$

t_d : Rain duration

k_h : Constant depends on soil characteristics and vegetative cover

2. Surface of entry : at the soil surface, the impact of raindrops causes the fines in the soil to be displaced and these in turn can clog the pore spaces in the upper layers of the soil. Thus a surface covered with grass and other vegetation which can reduce this process this process has pronounced influence on the value of f_p .

3. Fluid Characteristics : water infiltrating into the soil will have many impurities, both in solution and in suspension. The turbidity of water, especially the clay and colloid content is an important factor and such suspended particles block the fine pores in the soil and reduce its infiltration capacity.

3.9. Infiltration Capacity :

In hydrological calculations inolving floods, it is found convenient to use a constant value of infiltration rate for the duration of the storm. The defined average infiltration rate is called infiltration index and two types of indices are in common use :

1. **Φ - index : the average rainfall above which the rainfall volume is equal to the runoff volume**

$$\emptyset = \frac{P - R_d}{t_e} \quad .... (10)$$

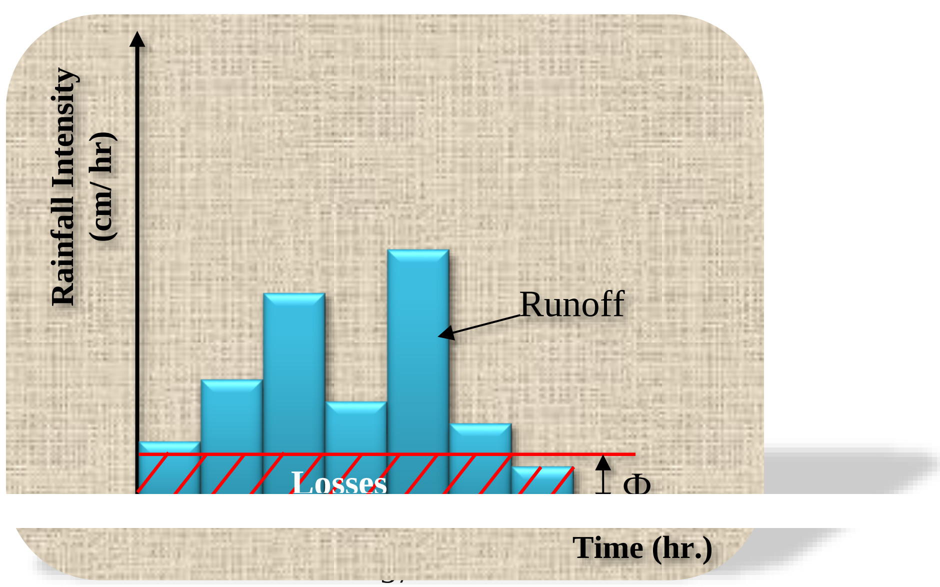
In which total rainfall (P) is equal to :

$$P = \sum_1^N I_i * \Delta t \quad .... (10.1)$$

I_i : intensity of rainfall in i_{th} pulse its time Δt
And R_d is the total direct runoff is calculated from the following equation :

$$R_d = \sum_1^M (I_i - \emptyset)\Delta t \quad .... (10.2)$$

t_e : duration of excess rainfall



Example (4) : A storm with 10 cmof precipitation produced a direct runoff of 5.8 cm. The duration of the rainfall was 16 hours and its time distribution is given below. Estimate the Φ index of the storm.

Time from start (hr)	0	2	4	6	8	10	12	14	16
Cumulative Rainfall (cm)	0	0.4	1.3	2.8	5.1	6.9	8.5	9.5	10

Solution :

Time from start (hr)	2	4	6	8	10	12	14	16
Cumulative Rainfall (cm)	0.4	1.3	2.8	5.1	6.9	8.5	9.5	10.0
Incremental rain (cm)	0.4	0.9	1.5	2.3	1.8	1.6	1.0	0.5
Intensity of rain I_i (cm/hr)	0.2	0.45	0.75	1.15	0.9	0.8	0.5	0.25

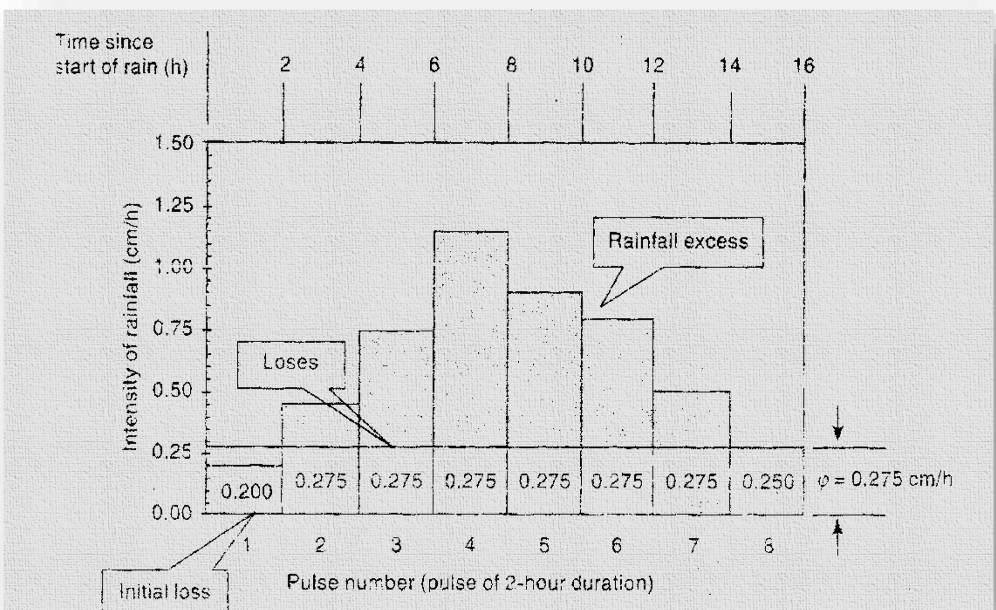
Here, duration of rainfall D = 16 hr , Δt = 2 hr and N = 8

Trial 1 :

Assume M = 8 , Δt = 2 hr and hence t_e = M *Δt = 16 hr

Runoff R_d = 5.8 cm = $\sum_1^8 (I_i - \emptyset)\Delta t$

5.8 = [(0.2*2) + (0.45*2) + (0.75*2) + (1.15*2) + (0.9*2) + (0.8*2) + (0.5*2) + (0.25*2)] – 16 Ø = 10 – 16 Ø , then $\emptyset = \frac{4.2}{16} = 0.263\ cm/hr$



Not OK , hence use M = 6 in the next trial

Trial 2 :

Assume M = 6 , Δt = 2 hr and hence t_e = M *Δt = 12 hr

Runoff R_d = 5.8 cm = $\sum_1^6 (I_i - \emptyset)\Delta t$

5.8 = [(0.45*2) + (0.75*2) + (1.15*2) + (0.9*2) + (0.8*2) + (0.5*2)] – 12 ∅

= 9.1 – 12 ∅

$\emptyset = \frac{3.3}{12} = 0.275\ cm/hr$ **(OK)**

---

2. W- index : is an attempt to refine the Φ - index the initial losses are separated from the total abstractions and an average value of infiltration rate, called W – index, is defined as :

$$W = \frac{P - R - I_a}{t_e} \quad \text{.... (11)}$$

P : total storm precipitation (cm)
R : total storm runoff (cm)
I_a : Initial losses (cm)
t_e : duration of the rainfall excess

Chapter Four

Runoff

4.1. Runoff : the draining or flowing off of precipitation from a catchment area through a surface channel. It thus represents the output from the catchment in a given unit of time.

Flows from several small channels join bigger channels and flows from these in turn combine to form a larger stream, and so on, till the flow reaches the catchment outlet. The flow in this mode where it travel all the time over the surface as overland flowand through the channels as open channel flow and reaches the catchment outlet is called surface.

The runoff is classified into two categories :

1. Direct Runoff : it is that part of the runoff which enters the stream immediately after the rainfall. It includes surface runoff, prompt interflow and rainfall on the stream.
2. Base Flow : the delayed flow that reaches a stream essentially as ground water flow is called base flow.

4.2. Natural Flow (Virgin Flow) : when stream flow in its natural condition, i.e. without human intervention. Such a stream flow unaffected by works of man, such flows is called natural flow or virgin flow.

$$R_N = (R_o - V_r) + V_d + E + E_x + \Delta S \quad \text{…. (1)}$$

R_N :Natural flow volume in time Δt

R_o :Observed flow volume in time Δt at the terminal site

V_r : Volume of return flow from irrigation, domestic water supply and industrial use

V_d : Volume diverted out of the stream for irrigation, domestic water supply and industrial use

E : Net evaporation losses from reservoirs on the stream

E_x : Net export of water from the basin

ΔS : change in the storage volumes of water storage bodies on the stream.

Example (1) : The following table gives values of measured discharges at a steam gauging site in a year. Upstream of the gauging site, a weir built across the stream diverts 3 Mm^3 and 0.5 Mm^3 of water per month for irrigation and for use in an industry respectively. The return flows from the irrigation is estimated as 0.8 Mm^3 reaching the stream upstream of the gauging site and 0.3 Mm^3 from industry. Estimate the natural flow. If the catchment area is 180 km^2 and the average annual rainfall is 185 cm. determine the runoff-rainfall ratio.

Month	1	2	3	4	5	6	7	8	9	10	11	12
Gauged flow (Mm^3)	2	1.5	0.8	0.6	2.1	8	18	22	14	9	7	3

Solution :

Here E, E_x and ΔS are assumed to be insignificant and zero value.

$V_r = 0.8 + 0.3 = 1.1\ Mm^3$

$V_d = 3 + 0.5 = 3.5\ Mm^3$

Month	1	2	3	4	5	6	7	8	9	10	11	12
V_s	2	1.5	0.8	0.6	2.1	8	18	22	14	9	7	3
V_d	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5
V_r	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
Rv	4.4	3.9	3.2	3	4.5	10.4	20.4	24.4	16.4	11.4	9.4	5.4

Σ Rv = 116.8 Mm^3

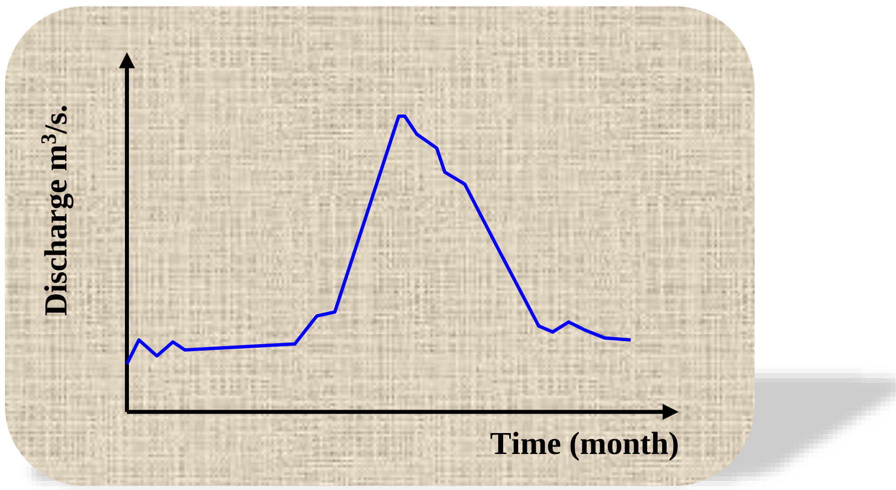
Annual Runoff = 116.8 * 10^6 / 120 *10^6 = 0.973 m. = 97.3 cm.

Runoff Coefficient = Runoff / Rainfall = 97.3 / 185 = 0.526

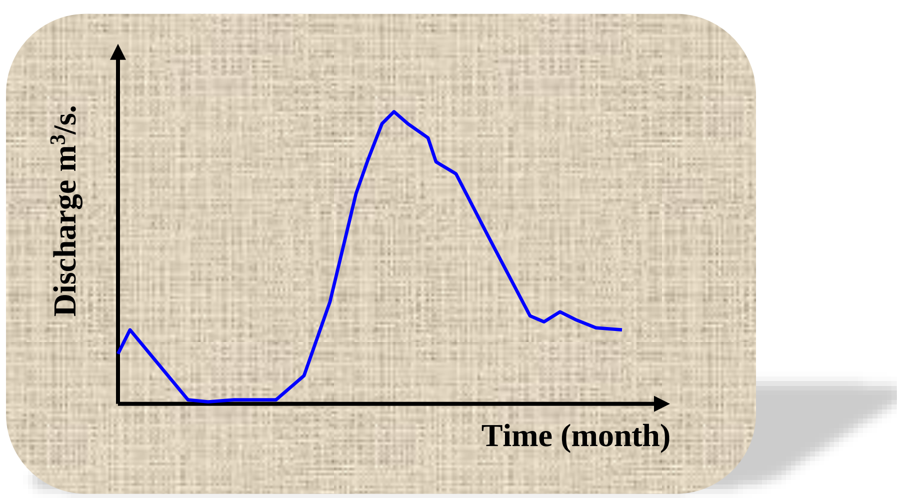
4.3. Runoff Characteristics of Streams :

A study of the annual hydrographs of streams enables one to classify streams into three classes as :

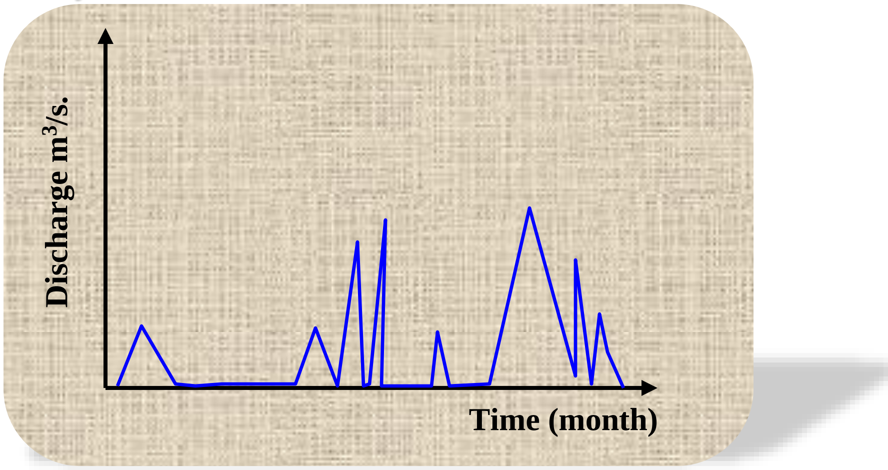
1. Perennial streams : always carries some flow. There is considerable amount of groundwater flow throughout the year. Even during the dry seasons, the water table will be above the bed of the stream.



2. Intermittent streams : has limited contribution from the groundwater. During the wet season, the water table is above the stream bed and there is a contribution of the base flowto the stream flow. However, during dry seasons, the stream remains dry for the most part of the dry months.



2. Ephemeral stream : which does not have any base flow contribution. The stream becomes dry soon after the end of the storm flow.



The stream characteristics of a stream depend upon :

1. The rainfall characteristics, such as magnitude, intensity, distribution according to time and space and its variability.
2. Catchment characteristics such as soil, land use / cover, slope, geology, shape and drainage density.
3. Climatic factors which influence evaporation.

4.4. Runoff Volume (Yield) :

The total quantity of surface water that can be expected in a given period from a stream at the outlet of its catchment.

The yield of a catchment Y in a period Δt could be expressed by water balance equation as :

$$Y = R_N + V_r = R_o + A_b + \Delta S \quad \ldots. (2)$$

R_N :Natural flow volume in time Δt

R_o :Observed flow volume in time Δt at the terminal site

V_r : Volume of return flow from irrigation, domestic water supply and industrial use

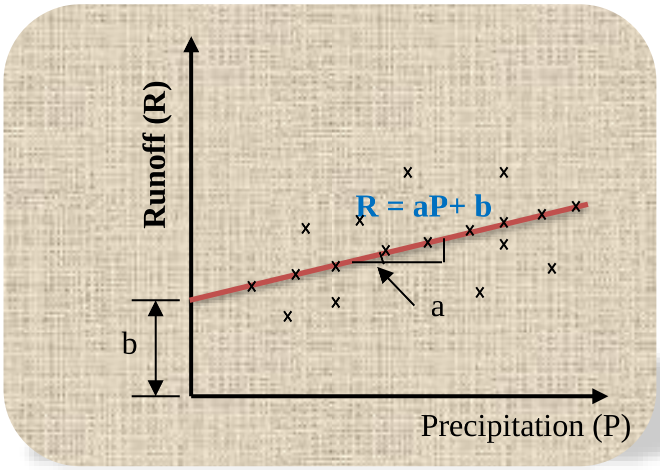
A_b : Abstraction from in time Δt for multi purposes such as water supply, irrigation, evaporation losses …..etc.

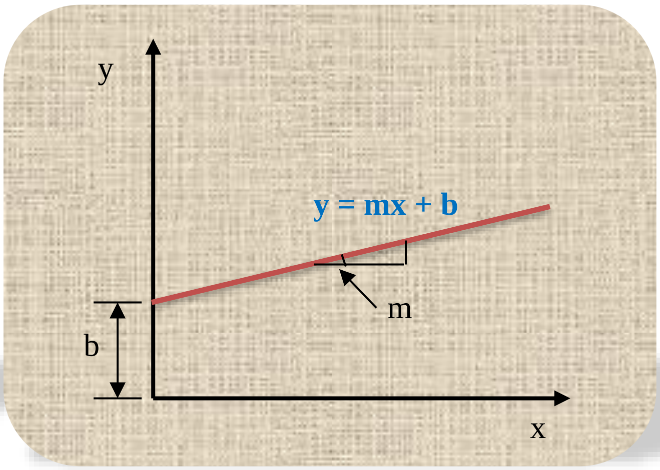
ΔS : change in the storage volumes of water storage bodies on the stream.

4.5. Yield Estimation Methods :

There are many methods for yield estimation such as :

4.5.1.Rainfall – Runoff Correlation





$$R = a\,P + b \quad \ldots\ldots\ldots\ (3)$$

$$a = \frac{N\,(\sum PR) - (\sum P)(\sum R)}{N\,(\sum P^2) - (\sum P)^2} \quad \ldots\ldots\ldots\ (4)$$

$$b = \frac{\sum R - a \sum P}{N} \quad \ldots\ldots\ldots\ldots\ldots\ (5)$$

N : number of observation sets R and P

The coefficient of correlation, r, can be calculated as :

$$r = \frac{N\,(\sum PR) - (\sum P)(\sum R)}{\sqrt{[N\,(\sum P^2) - (\sum P)^2] * [N\,(\sum R^2) - (\sum R)^2]}} \quad \ldots\ldots\ldots\ (4)$$

Note :

If ($0 \le r \le 1$) , then R can have only positive correlation with P.

If ($0.6 \le r \le 1$), then R has a good correlation with P.

Example (2) : Annual rainfall and runoff values (cm) of a catchment spanning a period of 21 years are given below. Analyze the data to :

a) Estimate the 75% and 50% dependable annual yield of the catchment

b) To develop a linear correlation equation to estimate annual runoff volume for a given annual rainfall value.

Year	P(cm)	R(cm)	Year	P(cm)	R(cm)
1975	118	54	1986	75	17
1976	98	45	1987	107	32
1977	112	51	1988	75	15
1978	97	41	1989	93	28
1979	84	21	1990	129	48
1980	91	32	1991	153	76
1981	138	66	1992	92	27
1982	89	25	1993	84	18
1983	104	42	1994	121	52
1984	80	11	1995	95	26
1985	97	32			

Solution :

Year	P (cm)	R (cm)	P^2	R^2	PR	m	Sorted R	Exceedance Probability
1975	118	54	13924	2916	6372	1	76	0.045
1976	98	45	9604	2025	4410	2	66	0.091
1977	112	51	12544	2601	5712	3	54	0.136
1978	97	41	9409	1681	3977	4	52	0.182
1979	84	21	7056	441	1764	5	51	0.227
1980	91	32	8281	1024	2912	6	48	0.273
1981	138	66	19044	4356	9108	7	45	0.318
1982	89	25	7921	625	2225	8	42	0.364
1983	104	42	10816	1764	4368	9	41	0.409
1984	80	11	6400	121	880	10	32	
1985	97	32	9409	1024	3104	11	32	
1986	75	17	5625	289	1275	12	32	0.545
1987	107	32	11449	1024	3424	13	28	0.591
1988	75	15	5625	225	1125	14	27	0.636
1989	93	28	8649	784	2604	15	26	0.682
1990	129	48	16641	2304	6192	16	25	0.727
1991	153	76	23409	5776	11628	17	21	0.773
1992	92	27	8464	729	2484	18	18	0.818
1993	84	18	7056	324	1512	19	17	0.864
1994	121	52	14641	2704	6292	20	15	0.909
1995	95	26	9025	676	2470	21	11	0.955
SUM	**2132**	**759**	**224992**	**33413**	**83838**			

a = 0.7938 , b = - 44.44 , $R = 0.7938\,P - 44.44$

r = 0.949 (R has a good and positive correlation with P)

4.5.2. Empirical Equation :

4.5.2.1. Khosla's formula :

Khosla (1960) analysed the rainfall, runoff and temperature data for various catchments in India and USA to arrive to an ampirical relationship between rainfall and runoff :

$$R_m = P_m - L_m \;\dots.\; (5)$$

$$L_m = 0.48\, T_m \;\dots.\; (6) \qquad T_m > 4.5o\ C$$

R_m : Monthly surface runoff (cm) ($R_m \geq 0$)

P_m : Monthly rainfall (cm)

L_m : Monthly Losses (cm)

T_m : Mean monthly temperature of the catchment in ^oC

For $T_m \leq 4.5^o$ C, the loss L_m may provisionally be assumed as

T(^{o}C)	4.5	-1	- 6.5
L_m (cm)	2.77	1.78	1.52

Month	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Temp ^{o}C	12	16	21	27	31	34	31	29	28	29	19	14
P_m (cm)	4	4	2	0	2	12	32	29	16	2	1	2

Example (3) : For a catchment, the mean monthly temperatures are given. Estimate the annual runoff and annual runoff coefficient by Khosla's formula.

Solution :

From the table above, all temperatures are above 4.5^{o}C :

$L_m = 0.48\, T_m$

Month	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Jan.
P_m (cm)	4	4	2	0	2	12	32	29	16	2	1	2	4
T_m ^{o}C	12	16	21	27	31	34	31	29	28	29	19	14	12
L_m (cm)	4	4	2	0	2	12	14.9	13.9	13.4	2	1	2	4
R_m (cm)	0	0	0	0	0	0	17.1	15.1	2.6	0	0	0	0

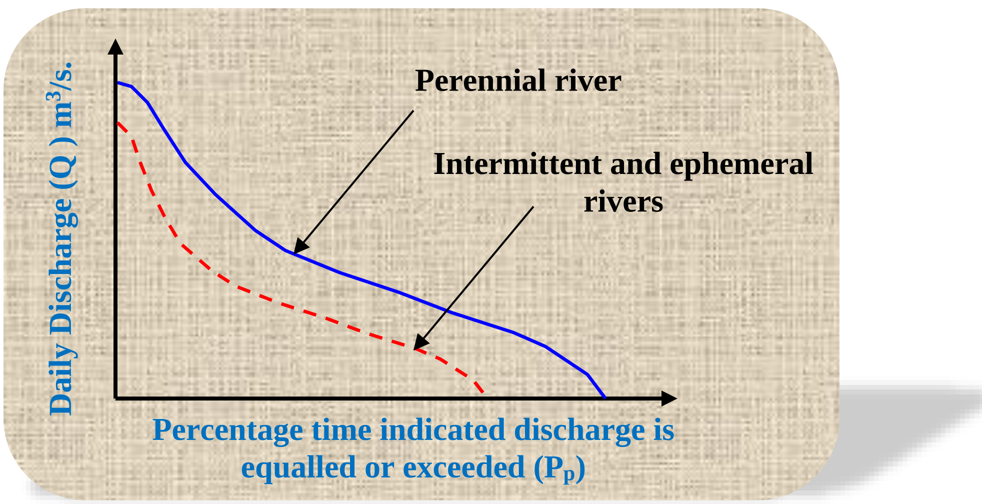
Total annual runoff = 34.8 cm.

Annual runoff coefficient = 34.8/116 = 0.3

4.6. Flow – Duration Curve :

A relationship between discharge against the percent of time in which the flow is equalled or exceeded. It is also known as Discharge – Frequency curve. If N number of data points are used in this listing, the plotting position of any discharge Q is :

$$P_P = \frac{m}{n+1} * 100 \;\ldots.\; (7)$$



4.6.1. Flow – Duration Curve Characteristics :

1. The slope of a flow – duration curve depends upon the interval of data selected.
2. The presence of a reservoir in a stream considerably modifies the flow – duration curve depending on the nature of flow regulation effect.
3. This curve when plotted on a log probability paper plots as a straight line at least over the central region. From this property, various coefficients expressing the variability of the flow in a stream can be developed for the description and comparison of different stream.
4. The chronological sequence of occurrence of the flow is masked in the flow – duration curve.
5. The flow – duration curve plotted on a log – log paper. It is useful in comparing the flow characteristics of different streams.

And some of important uses are :

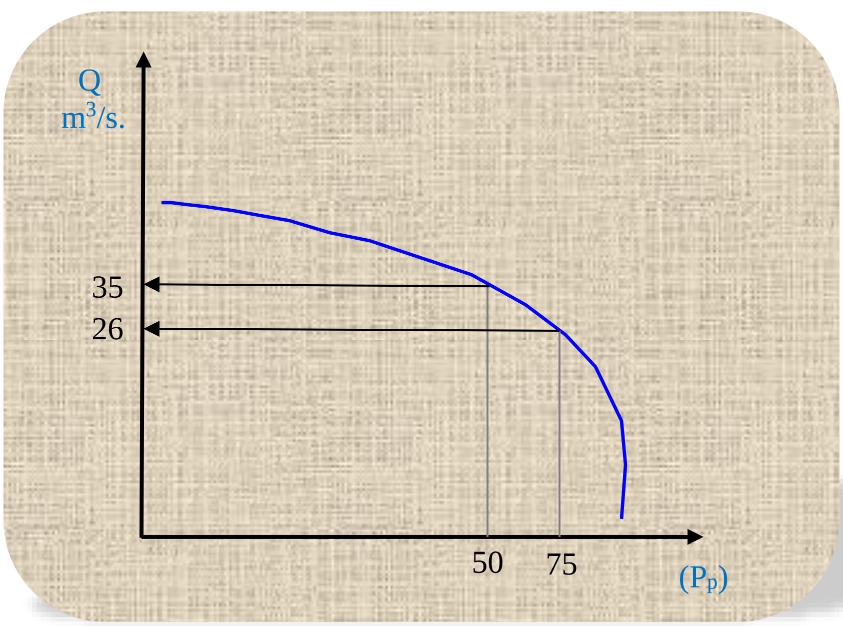
1. In evaluating various dependable flows in the planning of water resources engineering projects.
2. Evaluating the characteristics of the hydropower potential of a river.
3. Designing of drainage systems.
4. In flood – control studies.
5. Computing the sediment load and dissolved solids load of a stream.
6. Computing the adjacent catchments with a view to extend the stream flow data.

Example (4) : The daily low of a river for three consecutive years are shown in the table below. The table also contain the number of days of the flow belonged to the classes of discharges. Calculate the 50% and 75% dependable flows for the river.

Daily Mean Discharge (m^3/s)	No. of days flow in each class interval		
	1961 - 1962	1962 - 1963	1963 - 1964
140 - 120.1	0	1	5
120 - 100.1	2	7	10
100 - 80.1	12	18	15
80 - 60.1	15	32	15
60 - 50.1	30	29	45
50 - 40.1	70	60	64
40 - 30.1	84	75	76
30 - 25.1	61	50	61
25 - 20.1	43	45	38
20 - 15.1	28	30	25
15 - 10.1	15	18	12
10 - 5.1	5	0	0

Solution :

Daily Mean Discharge (m^3/s)	No. of days flow in each class interval			Total 1961-1964	Cumulative of Total (m)	
	1961 - 1962	1962 - 1963	1963 - 1964			
140 - 120.1	0	1	5	6	6	0.55
120 - 100.1	2	7	10	19	25	2.28
100 - 80.1	12	18	15	45	70	6.38
80 - 60.1	15	32	15	62	132	12.03
60 - 50.1	30	29	45	104	236	21.51
50 - 40.1	70	60	64	194	430	39.19
40 - 30.1	84	75	76	235	665	60.62
30 - 25.1	61	50	61	172	837	76.3
25 - 20.1	43	45	38	126	963	87.78
20 - 15.1	28	30	25	83	1046	95.35
15 - 10.1	15	18	12	45	1091	99.45
10 - 5.1	5	0	0	5	1096	99.91



From Curve :

$Q_{50} = 35\ m^3/s$

$Q_{75} = 26\ m^3/s$

4.7. Flow – Mass Curve :

Is a plot of the cumulative discharge volume against time plotted in a chronological order. It is also mathematically known as an integration for the hydrograph :

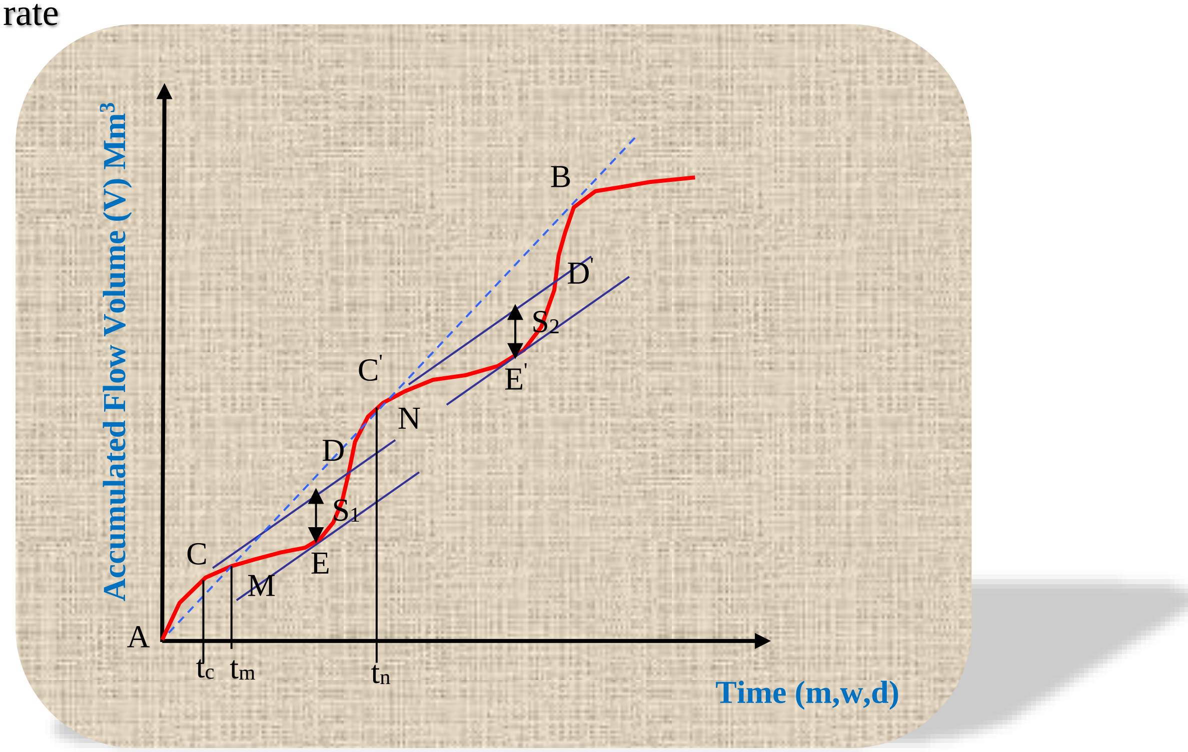
$$V = \int_{t_o}^{t} Q\,dt \;.... (8)$$

In which :

t_o : time at the beginning of the curve

t : time at the end of the curve

Q : discharge rate



Notes :

1. The slope of this curve at any point represents (Q = dv/dt) and this is equal to mean flow at any time.
2. The slope of the line AB represents the average flow along the period in which the curve was recorded.

4.8. Calculation of Storage Volume :

It is a cumulative difference between supply and demand volumes from the beginning of the dry season.

$$S = \Sigma V_S - \Sigma V_D \quad .....(9)$$

S : Maximum storage volume

ΣV_s : Supply volume

ΣV_D : Demand volume

The storage S, which is the maximum cumulative deficiency in any dry season is obtained as the maximum difference in the ordinate between mass curves of supply and demand.

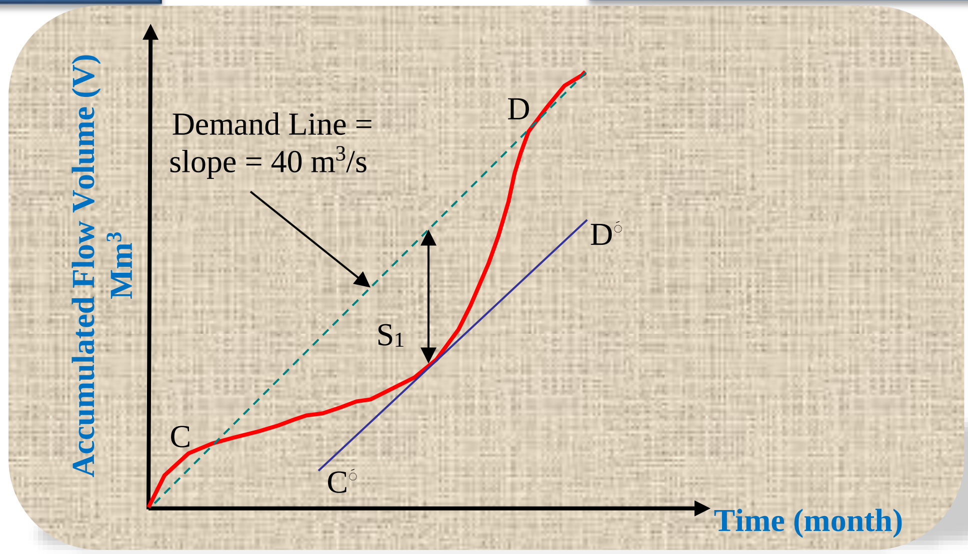
The minimum storage volume required by a reservoir is the largest of such S values over different dry periods.

Example (5) : The following table gives the mean monthly flows in a river during 1981. Calculate the minimum storage required to maintain a demand rate of 40 m^3/s.

Month	1	2	3	4	5	6	7	8	9	10	11	12
Mean Flow (m^3/s.)	60	45	35	25	15	22	50	80	105	90	80	70

Solution :

Month	Mean Flow m^3/s	Monthly Flow Volume (cumec.day)	Accumulated Volume (cumec. day)
1	60	1860	1860
2	45	1260	3120
3	35	1085	4205
4	25	750	4955
5	15	465	5420
6	22	660	6080
7	50	1550	7630
8	80	2480	10110
9	105	3150	13260
10	90	2790	16050
11	80	2400	18450
12	70	2170	20620



From graph :

For $Q_d = 40\ m^3/s.$ ⟹ $S_1 = 2100\ m^3/s.$ day

Example (6) : **Work out the previous example through arithmetic calculations without the use of mass curve. What is the maximum constant demand that can be sustained by this river ?**

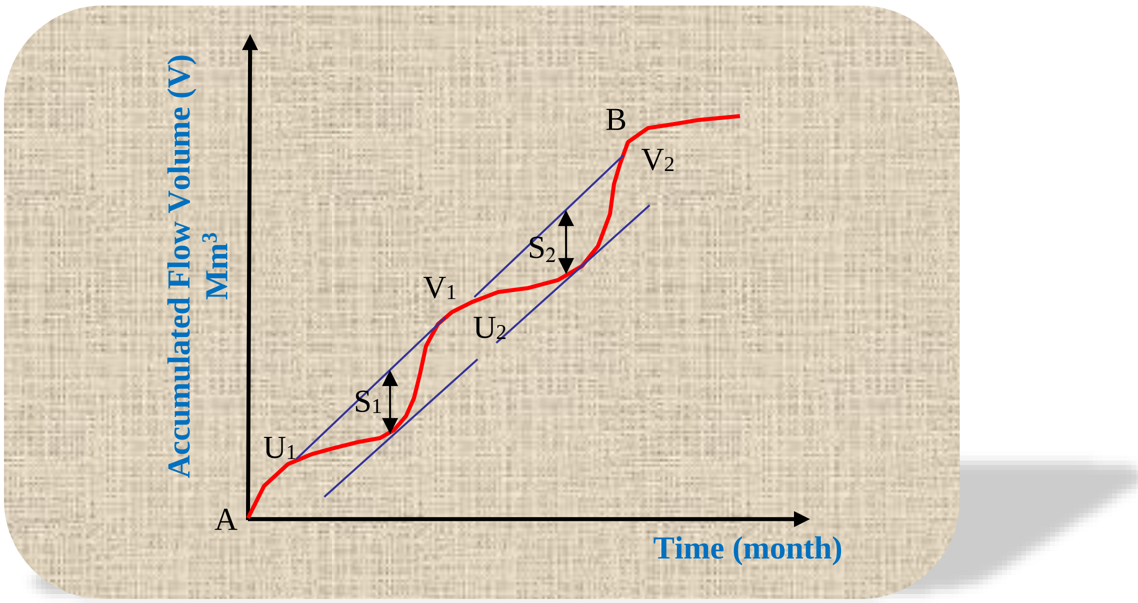
Solution :

Month	Mean Flow (m^3/s)	Monthly Flow Volume (cumec.day)	Demand Rate ($m^3/s.$)	Demand Volume (cumec.day)	Departure (col.3 – col.5)	Cumulative Excess Demand Volume (cumec.day)	Cumulative Excess Inflow Volume (cumec.day)
1	60	1860	40	1240	620		620
2	45	1260	40	1120	140		760
3	35	1085	40	1240	-155	-155	
4	25	750	40	1200	-450	-605	
5	15	465	40	1240	-755	-1380	
6	22	660	40	1200	-540	-1920	
7	50	1550	40	1240	310		310
8	80	2480	40	1240	1240		1550
9	105	3150	40	1200	1950		3500
10	90	2790	40	1240	1550		6050
11	80	2400	40	1200	1200		7250
12	70	2170	40	1240	930		8180

Then, the maximum demad (minimum storge) from column 7 is equal to 1920 m^3/s.day.

4.9. Calculation of Maintainable Demand :

Determining the maximum demand rate that can be maintained by a given storage volume.



The following salient points in the use of the mass curve are worth noting :

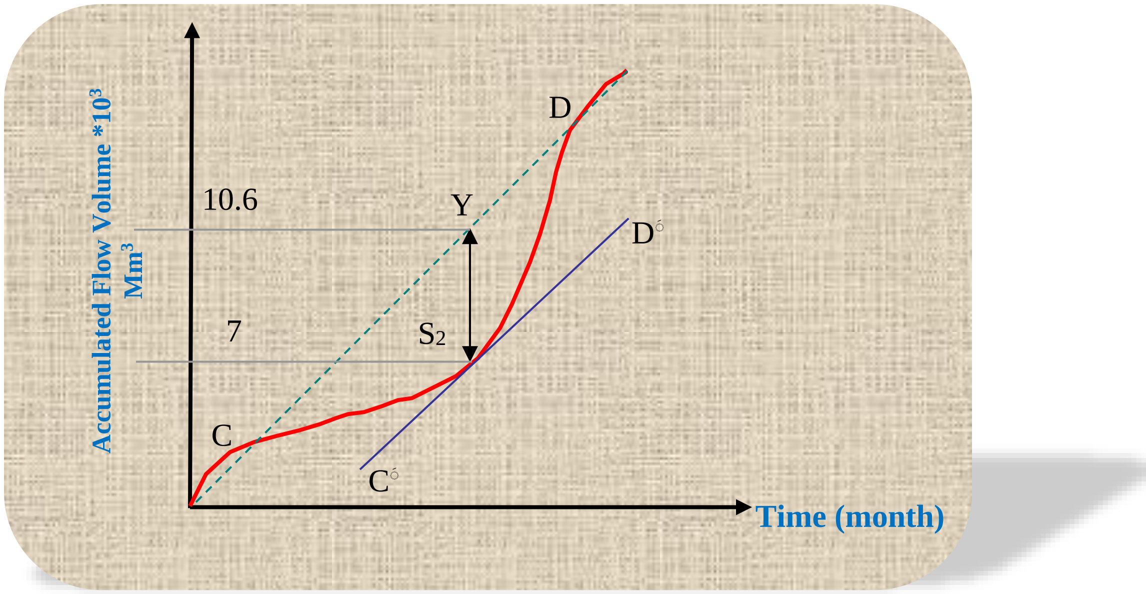
1. The vertical distance between two successive tangents to a mass curve at the ridges (points v_1 and u_2 in the figure above) represent the water wasted over the spillway.
2. A demand line must intersect the nass curve if the reservoir is to refill. Nonintersection of the demand line and mass curve indicates insufficient flow.

Example (7) : Using the mass curve of the previous example, obtain the maximum uiform rate that can be maintained by a storage of 3600 m^3/s days.

Solution :

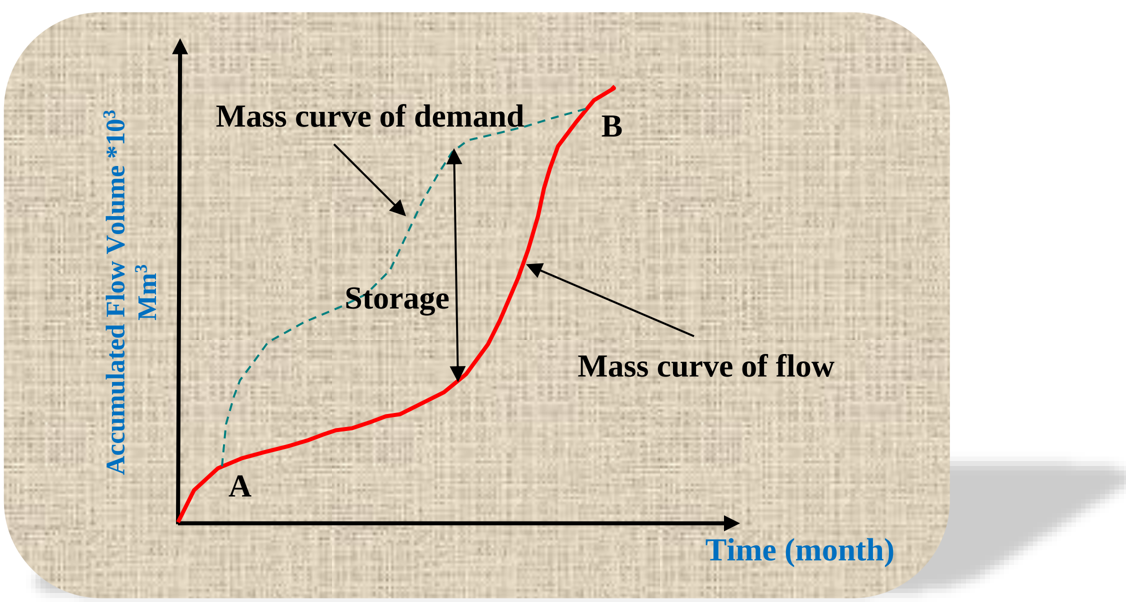
1. A vertical distance (XY) of 3600 c.day is drawn from approximate lowest position in the dip of the mass curve
2. A line passing through Y and tangential to the hump of the mass curve at C is drawn (Line CYD).

3. The slope of the line CYD corresponding to the final location of XY is the required demand rate. In this example, this rate is found to be 50 m^3/s.



4.10. Variable Demand :

The variation in the demand rate to meet the various end uses, such as irrigation, power and water supply needs.



Note : The reservoir is full at the points A & B.

Example (8) : For a proposed reservoir, the following data were calculated. The prior water rights required the release of natural flow or 5 m^3/s. whichever is

less. Assuming an average reservoir area of 20 km^2, estimate the storage required to meet these demands. (Assume the runoff coefficient of the area submerged by the reservoir = 0.5).

Month	Mean Flow (m^3/s)	Demand (Mm^3)	Monthly Evaporation (cm)	Monthly Rainfall (cm)
1	25	22	12	2
2	20	23	13	2
3	15	24	17	1
4	10	26	18	1
5	4	26	20	1
6	9	26	16	13
7	100	16	12	24
8	108	16	12	19
9	80	16	12	19
10	40	16	12	1
11	30	16	11	6
12	30	22	17	2

Solution :

Prior right release = $5 * 30.4 * 8.64 * 10^4 = 13.1\ Mm^3$ when $Q > 5\ m^3/s$.

Evaporation volume = $\frac{E}{100} * 20 * 10^6 = 0.2\ Mm^3$

Rainfall volume = $\frac{P}{100} * (1- 0.5) * 20 * 10^6 = 0.1\ P\ Mm^3$

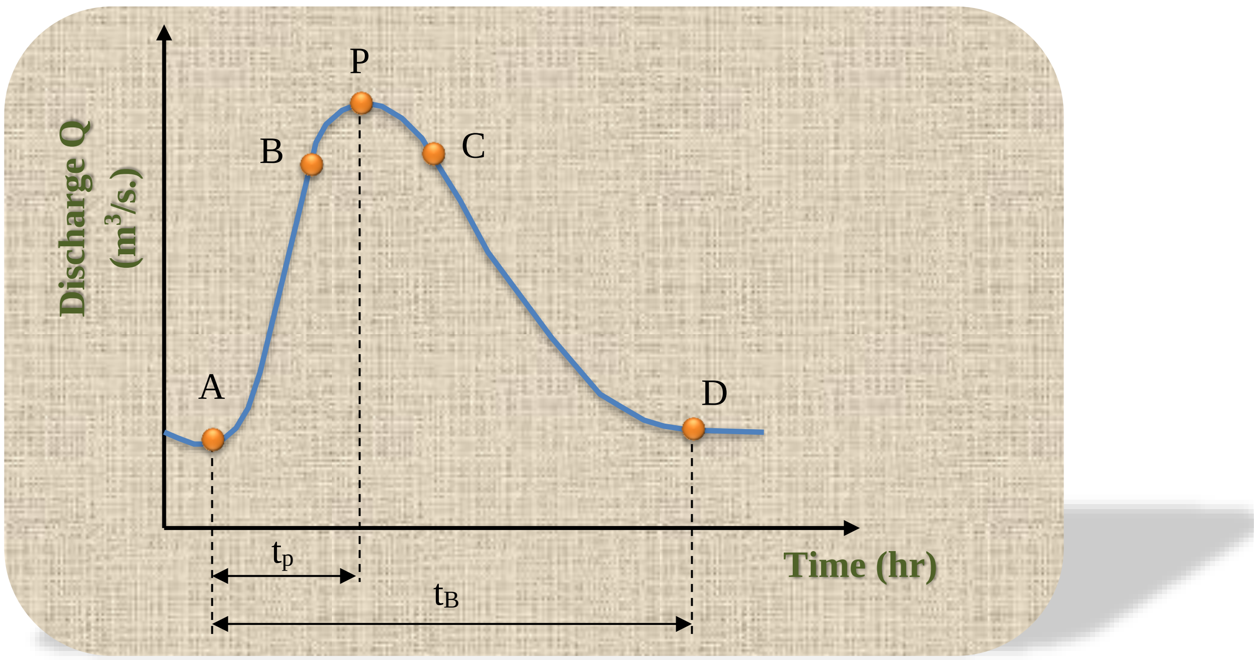
Month	Inflow Volume (Mm^3)	Demand (Mm^3)	Withdrawal Prior Rights (Mm^3)	Evaporation (Mm^3)	Rainfall (Mm^3)	Total Withdrawal (Mm^3)	Departure (Mm^3)	Cumulative Excess Demand (Mm^3)	Cumulative Excess Flow Volume (Mm^3)
January	67	22	13.1	2.4	-0.2	37.3	29.7	–	29.7
February	.484	23	13.1	2.6	-0.2	38.5	9.9	–	39.6
March	40.2	24	13.1	3.4	-0.1	40.4	-0.2	−0.2	–
April	25.9	26	13.1	3.6	-0.1	42.6	-16.7	-16.9	–
May	10.7	26	10.7	4	-0.1	40.6	-29.9	-46.8	–
June	23.3	26	13.1	3.2	-1.3	41.0	-17.7	-64.5	–
July	267.8	16	13.1	2.4	-2.4	29.1	23.87	–	238.7
August	289.3	16	13.1	2.4	-1.9	29.6	25.97	–	498.4
September	207.4	16	13.1	2.4	-1.9	29.6	17.78	–	676.2
October	107.1	16	13.1	2.4	-0.1	31.4	7.57	–	751.9
November	77.8	16	12.1	2.2	-0.6	30.7	47.1	–	799.0
December	80.4	22	13.1	3.4	-0.2	38.3	42.1	–	841.1

Maximum demand = 64.5 Mm^3

Chapter Five

Hydrograph

5.1. Hydrograph : **Measuring the stream flow over a specific catchment due to a storm of rainfall against time.**



The figure above represents storm hydrograph which results due to an isolated storm. It is also called as flood hydrograph.

5.2. Components of a Hydrograph :

1. Rising Limb : **also known as concentration curve represents the increase in discharge due to the gradual building up of storage in channels and over the catchment surface. The initial losses and high infiltration losses during the early period of a storm caused the discharge to rise rather slowly in the initial periods. As the storm continues more and more, flow from distant parts reach the basin outlet. Simultaneously, the infiltration losses also decrease with time. Thus under a uniform storm over the catchment, the runoff increases rapidly with time.**

2. Crest Segment : One of the most important parts of a hydrograph as it contains the peak flow. The peak flow occurs when the runoff from various parts of the catchment simultaneously contribute amounts to achieve the maximum amount of flow at the basin outlet.

3. Recession Limb : It is extends from the point of infiltration at the end of the crest segment (Point C) to the commencement of the natural groundwater flow (point D) represents the withdrawal of water from the storage built up in the basin during the earlier phases of the hydrograph.

4. Peak Time (t_p) : The time between point A to point P.

5. Base Time (t_B) : The time between point A to point B.

5.3. Hydrograph Phases :

1. Surface Runoff
2. Inter Flow
3. Base Flow

5.4. Factors Affecting Flood Hydrograph :

1. Shape of the Basin : the shape of the basin influences the time taken for water from the remote parts of the catchment to arrive at the outlet. Thus, the occurrence of the peak and hence the shape of the hydrograph are affected by the basin shape.

2. Size of the Basin : small basins behave different from the large ones in terms of the relative importance of various phases of the runoff phenomenon. In small catchment, the overland flow phase is predominant over the channel flow.

3. Slope of the Basin : the slope of the main stream controls the velocity of flow in the channel. As the recession limb of the hydrograph represents the depletion of storage, the stream channel slope will have a pronounced effect on this part of the hydrograph. Large stream slopes give rise to quicker depletion of storage and hence result in steeper recession limbs of hydrographs. This would obviously result in a smaller time base. The basin slope is important in small catchments

where the overland flow relatively more important. In such cases, the steeper slope of the catchment results in larger peak discharges.

4. Drainage Density : **drainage density may be defined as the ratio of the total channel length to the total drainage area. A large drainage density creates situation conducive for quick disposal of runoff down the channels. This fast response is reflected in a pronounced peaked discharge. In basins with smaller drainage densities, the overland flow is predominant and the resulting hydrograph is squat with a slowly rising limb.**

5. Land Use : **Vegetation and forests increase the infiltration and storage capacities of the soils. Further, they cause considerable retardance to the overland flow. Thus the vegtal cover reduces the peak flow. This effect is usually very pronounced in small catchments of area less than 150 km^2.**

7. **Climatic Factors :** **Among climatic factors the intensity, duration and direction of storm movement are the three important ones affecting the shape of a flood hydrograph. In very small catchment, the shape of the hydrograph can also be affected by the intensity. The effect of the duration is reflected in the rising limb and peak flow. If the storm moves from upstream to downstream of the catchment, there will be a quicker concentration of flow at the basin outlet.**

5.5. Recession Curve Equation :

Barnes (1940) showed that the recession of a storage can be expressed as :

$$Q_t = Q_o K_r^t \quad \ldots\ldots (1)$$

Qt : discharge at time t
Qo : initial discharge
Kr : recession constant

$$K_r = k_{rs} \cdot k_{ri} \cdot k_{rb} \quad \ldots\ldots (2)$$

Krs : recession constant for surface storage (0.05 – 0.2)
Kri : recession constant for interflow storage (0.5 -0.85)
Krb : recession constant for baseflow (0.85 – 0.99)

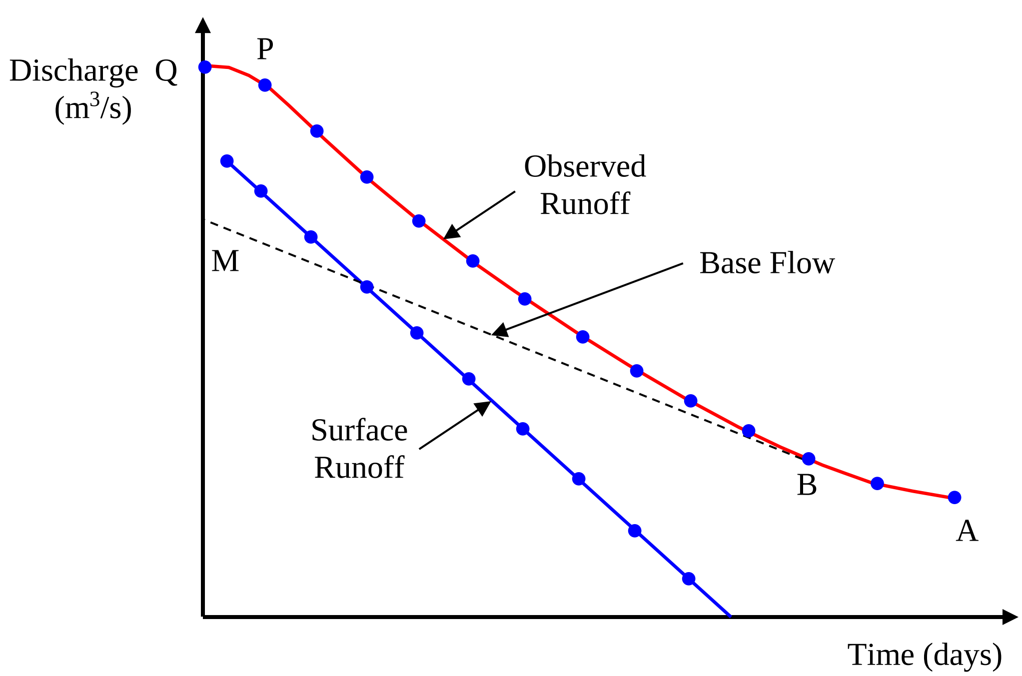
Example (1) : the recession limb of a flood hydrograph is given below. The time is indicated from the arrival of peak. Assuming the interflow component to be negligible, estimate the base flow and surface flow recession coefficients.

Time from Peak (day)	Discharge ($m^3/s.$)	Time from Peak (day)	Discharge ($m^3/s.$)
0	90	4	3.8
0.5	66	4.5	3
1	34	5	2.6
1.5	20	5.5	2.2
2	13	6	1.8
2.5	9	6.5	1.6
3	6.7	7	1.5
3.5	5		

Solution :

The data are plotted on a semi – log paper with discharge on the log – scale. The data points from t = 4.5 days to 7 day are seen to lie on straight line (line AB). This indicates that the surface flow terminates at t = 4.5 days.

$$Q_t / Q_o = K_{rb}^{\,t} \Longrightarrow \log K_{rb} = \frac{1}{t} \quad \log (Q_t / Q_o)$$



From figure above :

$Q_o = 6.6\ m^3/s.$, $t = 2$ days , $Q_t = 4\ m^3/s.$

$\log K_{rb} = \frac{1}{2} \log (4 / 6.6) \Longrightarrow K_{rb} = 0.78$

$Q_o = 26\ m^3/s.$, $t = 2$ days , $Q_t = 2.25\ m^3/s.$

$\log K_{rs} = \frac{1}{2} \log (2.25 / 26) \Longrightarrow K_{rs} = 0.29$

$K_r = 0.29 * 0.78 * 1 = 0.226$

5.6. Base Flow Separation Methods :

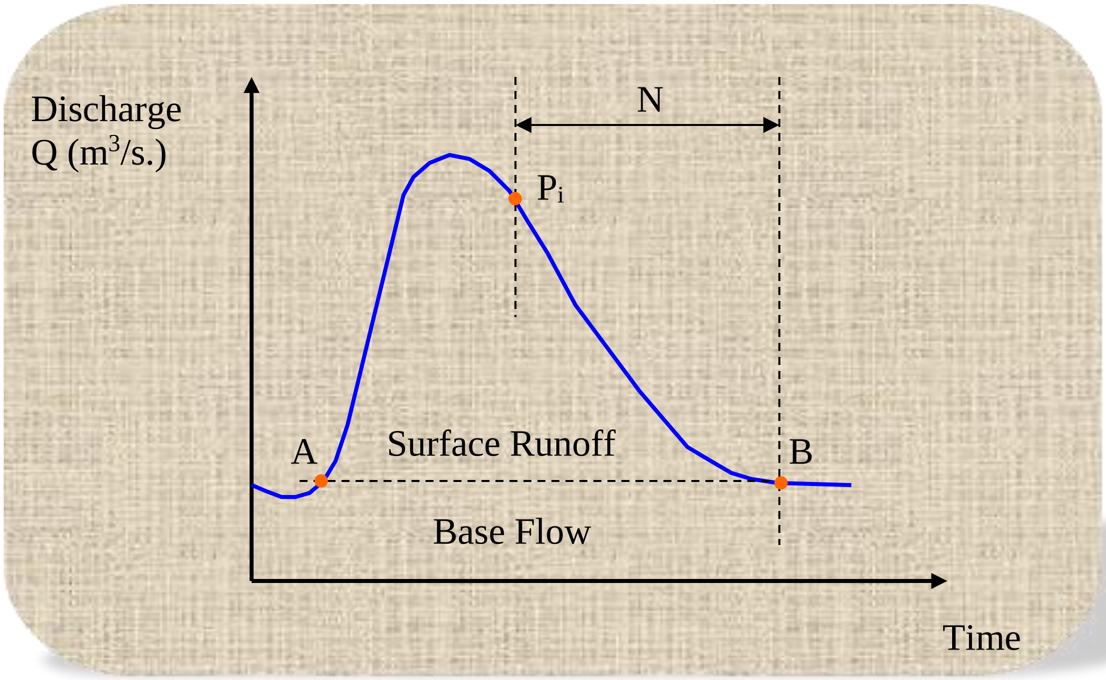
In many hydrograph analyses, a relationship between the surface – flow hydrograph and the effective rainfall (i.e. rainfall minus losses) is sought to be established. The surface flow hydrograph is obtained from the total storm hydrograph by sparating the quick response flow from the slow response runoff. It is usual to consider the interflow as a part of the surface flow in view of its quick response. Thus only the base flow is to be deducted from the total storm hydrograph to obtain the surface flow hydrograph. Ther are three methods of base flow separation that are in common use and asfollows :

I – Method I (Straight Line Method) :

In this method, the separation of the base flow is achieved by the joining with a straight line the beginning of the surface runoff (Point A) to a point of the recession limb representing the end of direct runoff (Point B) which can be indicated from the inflection point (P_i) with a distance equal to :

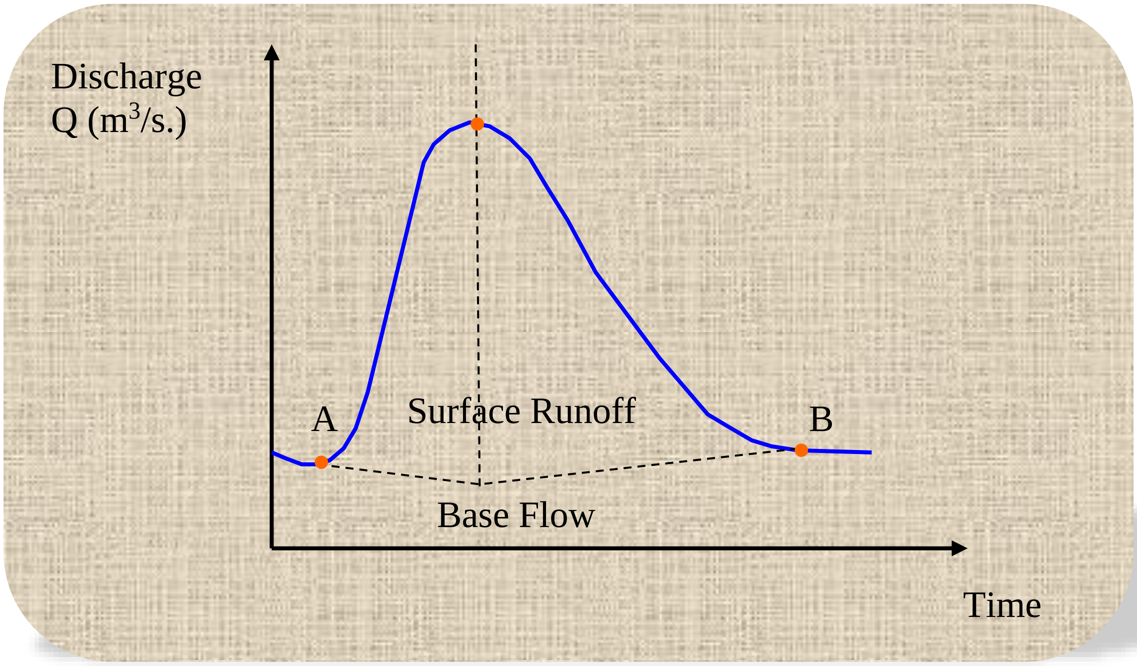
$$N = 0.83\ A^{0.2} \quad \text{....... (3)}$$

A : drainage area (km^2)



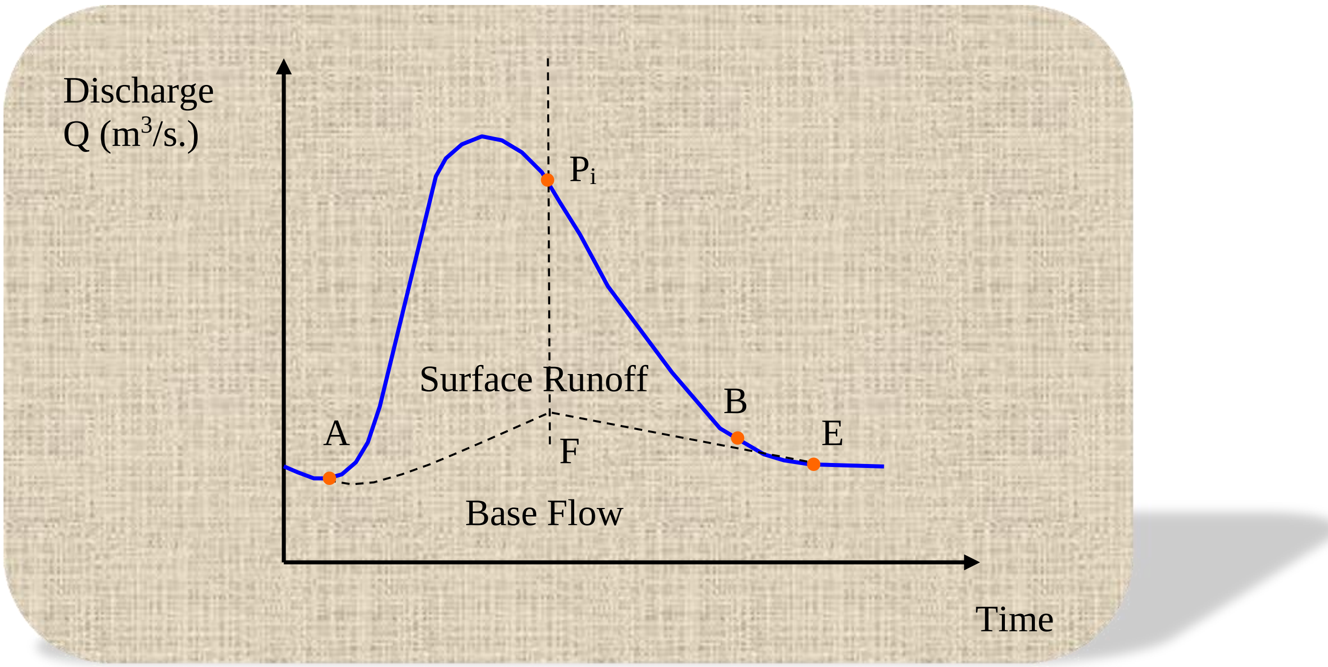
II – Method II :

In this method, the base flow curve existing prior to the commencement of the surface runoff is extended till it intersects the ordinate drawn at the peak (Point C). This point is joined to point B by a straight line. Segment AC and CB represents the border between base flow and surface runoff.



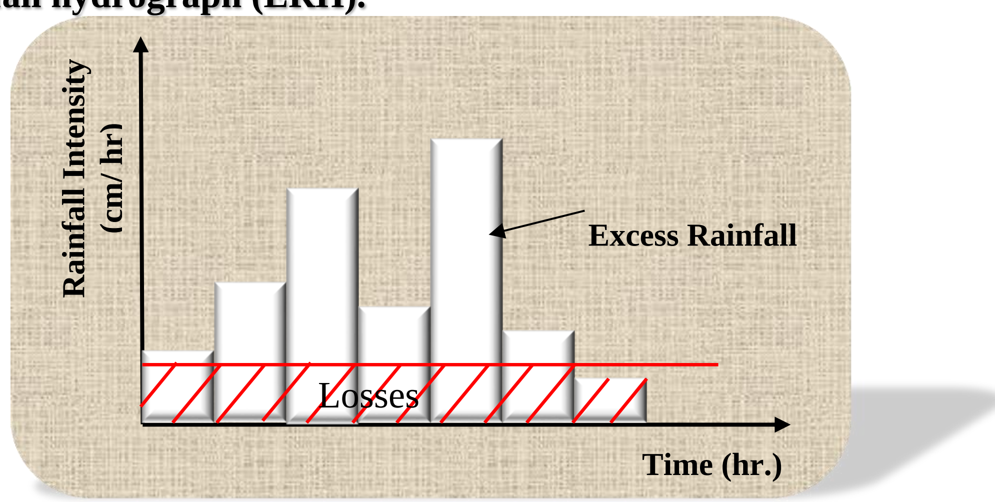
III – Method III :

In this method, the base flow recession curve after the depletion of the flood water is extended backwards till it intersects the ordinate at the point of inflection (Line EF). Line A and F are joined by an arbitrary smooth curve.



5.7. Effective Rainfall (ER) :

A part of rainfall that becomes direct runoff at the outlet of the watershed. It is thus the total rainfall in a given duration from which abstractions such as infiltration and initial losses are subtracted. The resulting hydrograph is known as effective rainfall hydrograph (ERH).

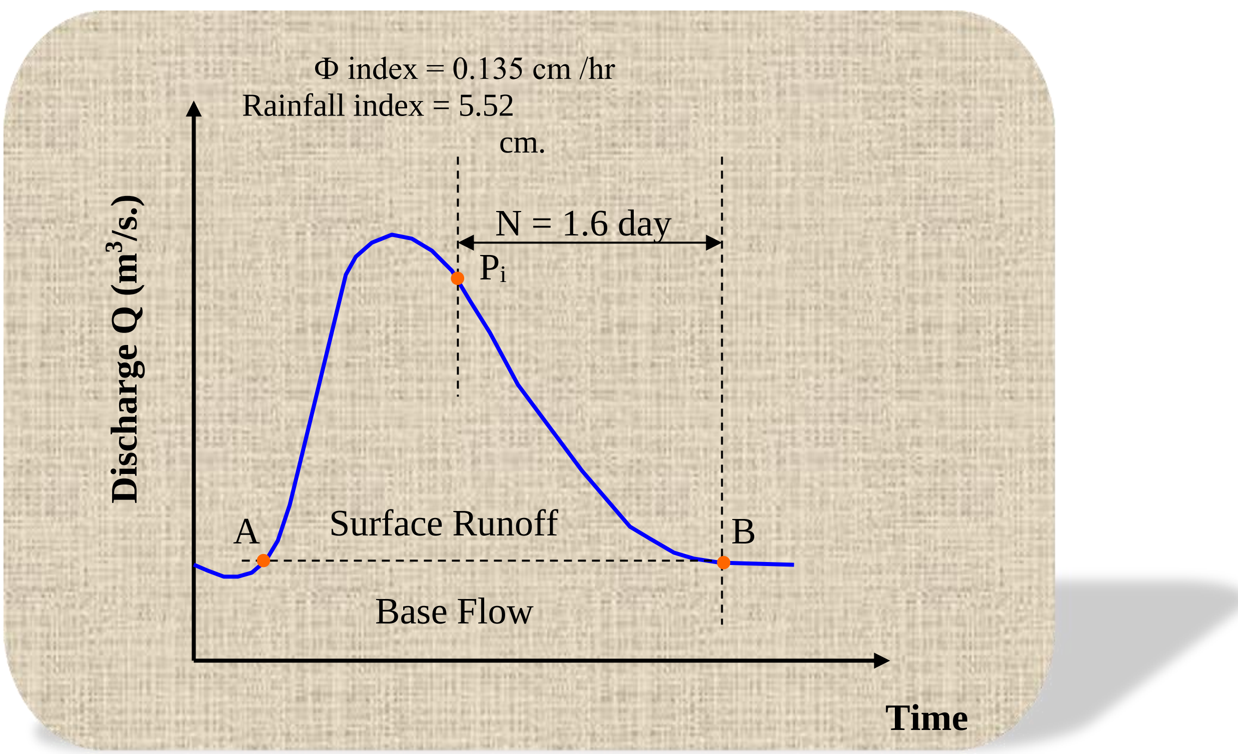


Note : **Each of (ERH and DRH) represent the same total quantity but in different units. The unit of ERH in (cm/hr) and when plotted against time, the result of the area under curve when multipled by the catchment area represents the total volume of direct runoff (area under DRH).**

Example (2) : Rainfall of magnitude 3.8 cm and 2.8 cm occuring on two consecutive 4 hr durations on a catchment of area 27 km^2 produced the following hydrograph of flow at the outlet of the catchment. Estimate the rainfall excess and Φ index.

Time from the start of rainfall (hr)	- 6	0	6	12	18	24	30	36	42	48	54	60	66
Observed flow (m^3/s)	6	5	13	26	21	16	12	9	7	5	5	4.5	4.5

Solution :



It is seen from figure above, the storm hydrograph has a base flow component. For using the simple straight line method of base flow separation :

$N = 0.83\,(27)^{0.2} = 1.6$ day = 38.5 hr
However, by inspection, DRH starts at t = 0 and ends at t = 48 hr and the peak point locate at t = 12 hr, then :
Time of N = 48 – 12 = 36 hr (more satisfactory)

Area of DRH = 6*60*60[0.5*8+0.5(8+21)+0.5(21+16)+0.5(16+11)+0.5(11+7)+0.5(7+4)+0.5(4+2)+0.5(2)]
$= 1.4904 \times 10^6\ m^3$

Depth of Runoff = Runoff vol./ Area = $1.4904 \times 10^6 / 27 \times 10^6$ = 5.52 cm. (Excess Rainfall)

Total Rainfall = 2.8 + 3.8 = 6.6 cm

Time of Duration = 8 hr

Φ index = (6.6 – 5.52) / 8 = 0.135 cm/hr

5.8. Unit Hydrograph :

The hydrograph of direct runoff resulting from one unit depth (1 cm) of rainfall excess occuring uniformly over the basin and at a uniform rate for a specified duration (D hours).

The definition of a unit hydrograph implies the following :

1. **The unit hydrograph represents the lumped response of the catchment to a unit rainfall excess of D – hduration to produce a direct runoff hydrograph. It relates only the direct runoff to the rainfall excess. Hence, the volume of water contained in the unit hydrograph must be equal to the rainfall excess. As 1 cm depth of rainfall excess is considered the area of the unit hydrograph is equal to a volume given by 1 cm over the catchment.**
2. **The rainfall is considered to have an average intensity of excess rainfall (ER) of 1/D cm/hr for the duration D – h of the storm.**
3. **The distribution of the storm is considered to be uniform all over the catchment.**

In general, the derivative of the DRH of the UH is based on the multiplying the recent coordinates by excess rainfall :

DRH Ordinates = UH Ordinates * ER

5.9. Unit Hydrograph Assumptions :

Two basic assumptions constitutes the foundations for the unit hydrograph theory. These are :

1. **The Time Invariance :** the first basic assumption is that the direct runoff response to a given effective rainfall in a catchment is time invariant. This implies that the DRH for a given ER in a catchment is always the same irrespective of when it occurs.
2. **Linear Response ;** the direct runoff response to the rainfall excess is assumed to be linear.

Example (3) : Given below are the ordinates of a 6- hr unit hydrograph for a catchment. Calculate the ordinates of the DRH due to a rainfall excess of 3.5 cm occuring in 6 hr.

Time (hr)	0	3	6	9	12	15	18	24	30	36	42	48	54	60	69
UH ordinates (m^3/s)	0	25	50	85	125	160	185	160	110	60	36	25	16	8	0

Solution :

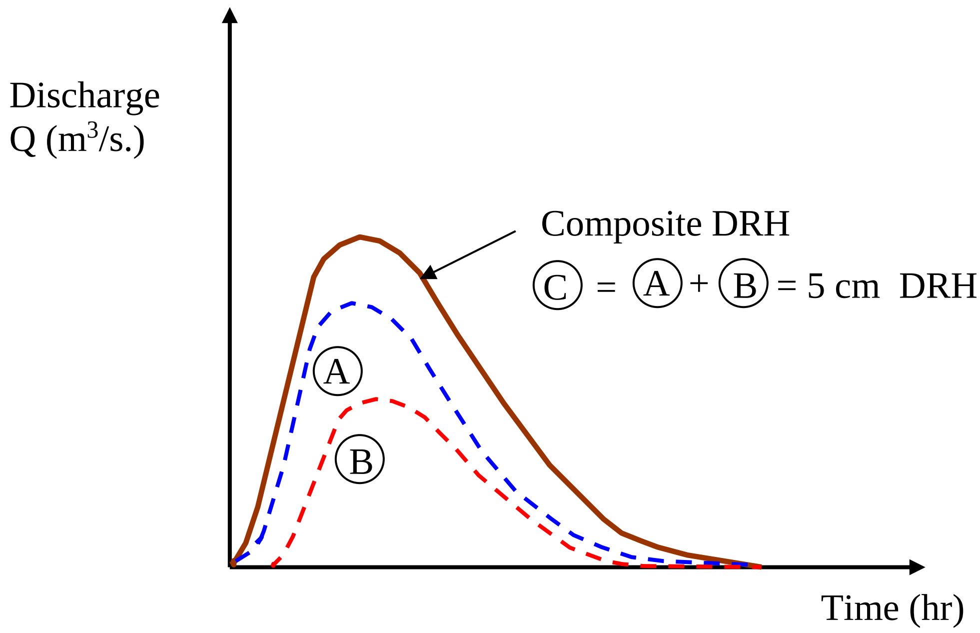
Time (hr)	0	3	6	9	12	15	18	24	30	36	42	48	54	60	69
UH – 6 hr ordinates (m^3/s)	0	25	50	85	125	160	185	160	110	60	36	25	16	8	0
Ordinates of 3.5 cm DRH (m^3/s)	0	87.5	175	297.5	437.5	560	647.5	560	385	210	126	87.5	56	28	0

Example (4) : Two storms each of 6- hr duration and having rainfall excess values of 3 and 2 cm respectively occur successively. The 2- cm ER rain follows the 3- cm rain. The 6- hr unit hydrograph for the catchment is the same as given in the previous example. Calculate the resulting DRH.

Solution :

Time (hr)	UH – 6 hr Ordinates (m3/s)	DRH – 3 cm (m3/s)	DRH – 2 cm (m3/s)	DRH – 5 cm (m3/s)
0	0	0	0	0
3	25	75	0	75
6	50	150	0	150
9	85	255	50	305
12	125	375	100	475
15	160	480	170	650
18	185	555	250	805
(21)	(172.5)	(517.5)	(320)	(837.5)
24	160	480	370	850
30	110	330	320	550

36	**60**	**180**	**220**	**400**
42	**36**	**108**	**120**	**228**
48	**25**	**75**	**72**	**147**
54	**16**	**48**	**50**	**98**
60	**8**	**24**	**32**	**56**
(66)	**(2.7)**	**(8.1)**	**(16)**	**(24.1)**
69	**0**	**0**	**(10.6)**	**(10.6)**
75	**0**			**0**



Example (5) : The ordinates of a 6- hr unit hydrograph of a catchment is given below :

Time (hr)	**0**	**3**	**6**	**9**	**12**	**15**	**18**	**24**	**30**	**36**	**42**	**48**	**54**	**60**	**69**
UH ordinates (m^3/s)	**0**	**25**	**50**	**85**	**125**	**160**	**185**	**160**	**110**	**60**	**36**	**25**	**16**	**8**	**0**

Derive the flood hydrograph in the catchment due to the storm given below :

Timefrom start of storm (hr)	**0**	**6**	**12**	**18**
Accumulated rainfall (cm)	**0**	**3.5**	**11**	**16.5**

Solution :

The effective rainfall hyetograph is calculated as in the following table :

Interval	1st 6 hrs.	2nd. 6 hrs.	3rd. 6 hrs.
Rainfall Depth (cm)	3.5	7.5	5.5
Loss at 0.25 cm/hr for 6 hr	1.5	1.5	1.5
Effective Rainfall (cm)	2	6	4

Time	Ordinates of UH	DRH – 2 cm	DRH – 6 cm	DRH – 4 cm	Ordinates of final DRH = col. 3 + col. 4 + col. 5	Base Flow	Flood Hydrograph = col. 6 + col. 7
1	2	3	4	5	6	7	8
0	0	0	0	0	0	15	15
3	25	50	0	0	50	15	65
6	50	100	0	0	100	15	115
9	85	170	150	0	320	15	335
12	125	250	300	0	550	17	567
15	160	320	510	100	930	17	947
18	185	370	750	200	1320	17	1337
(21)	(172.5)	(345)	(960)	(340)	(1645)	(17)	(1662)
24	160	320	1110	500	1930	19	1949
(27)	(135)	(270)	(1035)	(640)	(1945)	(19)	(1964)
30	110	220	960	740	1920	19	1939
36	60	120	660	640	1420	21	1441
42	36	72	360	440	872	21	893
48	25	50	216	240	506	23	529
54	16	32	150	144	326	23	349
60	8	16	96	100	212	25	237
66	(2.7)	(5.4)	(48)	(64)	(117)	(25)	(142)
69	-	-	-	-	-	-	-
72	0	0	16	32	48	27	75
75	-	-	-	-	-	-	-
78	0	0	0	(10.8)	(11)	27	49
81	0	0	0	0	0	27	27
84	0	0	0	0	0	27	27

5.10. Unit Hydrograph Derivation :

Is the process of finding coordinates of unit hydrograph by dividing the coordinates of the DRH on the value of the effective rain which is the value of the area under the curve of DRH and divided by the catchment area.

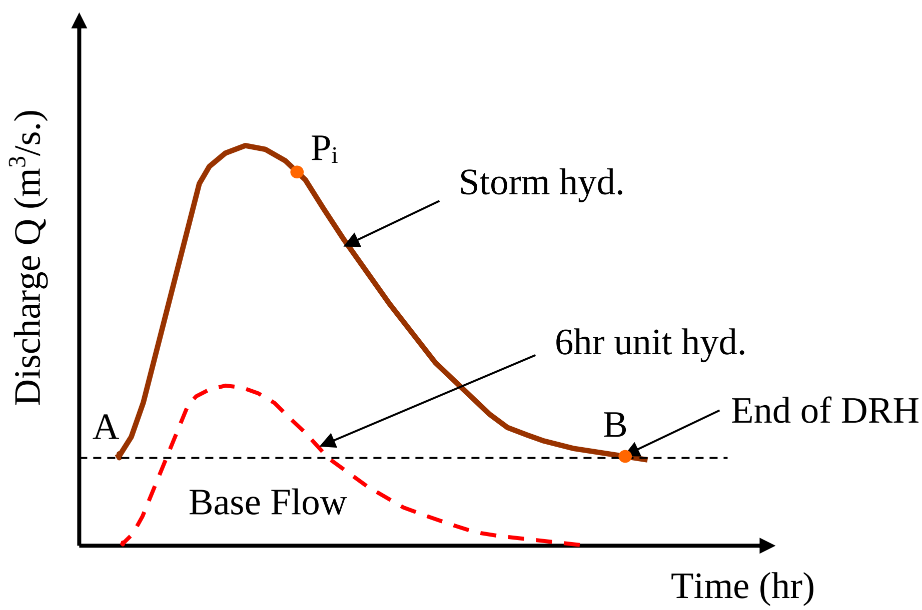
Flood hydrographs used in the analysis should be selected to meet the following desirable features with respect to the storms responsible for them :

1. **The storm should be isolated storms occuring individually.**
2. **The rainfall should be fairly uniform during the duration and should cover the entire catchment area.**
3. **The duration of the rainfall should be 1/5 to 1/3 of the basin lag.**
4. **The rainfall excess of the selected storm should be high. A range of ER values of 1 to 4 cm is sometimes preferred.**

Example (6) : Following are the ordinates of a storm hydrograph of a river draining a catchment area of 423 km^2 due to a 6- hr isolated storm. Derive the ordinates of a 6- hr unit hydrograph for the catchment.

Time from the start of storm (hr)	-6	0	6	12	18	24	30	36	42	48
Discharge (m^3/s)	10	10	30	87.5	115.5	102.5	85	71	59	47.5
Time from the start of storm (hr)	54	60	66	72	78	84	90	96	102	
Discharge (m^3/s)	39	31.5	26	21.5	17.5	15	12.5	12	12	

Solution :



A @ t = 0 hr. and B @ t = 90 hr.

Time from the start of storm (hr)	Ordinates of flood hydrograph (m^3/s)	Base Flow (m^3/s)	Ordinates of DRH (m^3/s)	Ordinates of 6- hr UH (Col.4 / 3)
1	2	3	4	5
−6	10	10	0	0
0	10	10	0	0
6	30	10	20	6.7
12	87.5	10.5	77	25.7
18	111.5	10.5	101	33.7
24	102.5	10.5	92	30.7
30	85	11	74	24.7
36	71	11	60	20
42	59	11	48	16
48	47.5	11.5	36	12
54	39	11.5	27.5	9.2
60	31.5	11.5	20	6.6
66	26	12	14	4.6
72	21.5	12	9.5	3.2
78	17.5	12	5.5	1.8
84	15	12.5	2.5	0.8
90	12.5	12.5	0	0
96	12	12	0	0
102	12	12	0	0

$\Sigma 587\ m^3/s$

Runoff Depth (ER) = $(587 * 6 * 3600) / 423 * 10^6 = 0.03$ m = 3 cm

Example (7) : a) The peak of flood hydrograph due to a 3- hr duration isolated storm in a catchment is 270 m^3/s. The total depth of rainfall is 5.9 cm. Assuming an average infiltration loss of 3 cm/hr and a constant base flow of 20 m^3/s. Estimate the peak of the 3- hr unit hydrograph of this catchment.

b) If the area of the catchment is 567 km^2, determine the base width of the 3- hr unit hydrograph by assuming it to be triangular in shape.

Solution :

a) Duration of rainfall excess = 3 hr , Total depth of rainfall = 5.9 cm

Total loss depth = 0.3 * 3 = 0.9 cm then ER = 5.9 – 0.9 = 5 cm

Peak of DRH = 270 – 20 = 250 m3/s

Peak of UH- 3 hr = 250 / 5 = 50 m3/s

b) Let B = base width of the 3- hr UH (hr)

Volume represented by the area of UH = 1 cm * area of catchment

= 0.5*B*60*60*50 = 567*106*0.01

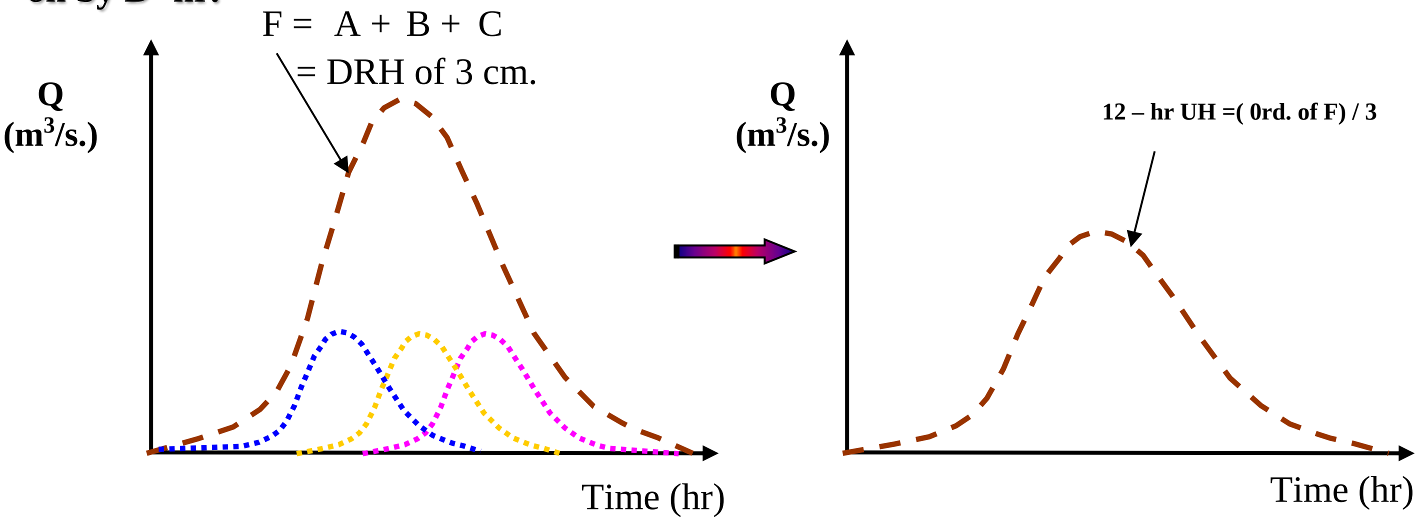
B = 63 hrs.

5.10. Unit Hydrograph For Different Durations :

There are many methods for derivation of unit hydrograph with nD- hr duration from another unit hydrograph its duration D- hr, and the most important methods are :

1. Super Position Method:

If a D- hr unit hydrograph is available, and it is desired to develop a unit hydrograph of nD- hr, where n is an integer, it is easily accomplished by superposing n unit hydrographs with each graph separated from the previous on by D- hr.



Example (8) : Given the ordinates of a 4- hr unit hydrograph as below. Derive the ordinates of a 12- hr unit hydrograph for the same catchment.

Solution :

Time (hr)	A	B lagged by 4- hr	C lagged by 8- hr	DRH– 3 cm (col. 2 + 3 + 4)	UH- 12 hr (col. 5 /3)
1	2	3	4	5	6
0	0	–	–	0	0
4	20	0	–	20	6.7
8	80	20	0	100	33.3
12	130	80	20	230	76.7
16	150	130	80	360	120
20	130	150	130	410	136.7
24	90	130	150	370	123.3
28	52	90	130	272	90.7
32	27	52	90	169	56.3
36	15	27	52	94	31.3
40	5	15	27	47	15.7
44	0	5	15	20	6.7
48	–	0	5	5	1.7
52	–	–	0	0	0

S – Curve Method :

It is desired to develop a unit hydrograph of duration mD, where m is a fraction, the method of superposition cannot be used. A different technique known as the S – curve method is adopted in such cases and this method is applicable for rational values of m.

Example (9) : Resolve the previous example using the S – curve method.

Solution:

Time (hr)	UH – 4 hr	S - Curve	S – Curve ordinates (col. 2 + 3)	S – Curve lagged by 12 - hr	Col. 4 – col. 5	Col. 6 / (4/12)
1	**2**	**3**	**4**	**5**	**6**	**7**
0	0	0	0	–	0	0
4	20	0	20	–	20	6.7
8	80	20	100	–	100	33.3
12	130	100	230	0	230	76.7
16	150	230	380	20	360	120
20	130	380	510	100	410	136.7
24	90	510	600	230	370	123.3
28	52	600	652	380	272	90.7
32	27	652	679	510	169	56.3
36	15	679	694	600	94	31.3
40	5	694	699	652	47	15.7
44	0	699	699	679	20	6.7
48	–	699	699	694	5	1.7
52	–	–	699	699	0	0

Example (10) : Ordinates of UH- 4hr are given in the table below. Use these ordinates and derive ordinates of UH- 2 hr for the same catchment.

Solution :

Time (hr)	UH – 4 hr	S - Curve	S – Curve ordinates (col. 2 + 3)	S – Curve lagged by 2 - hr	Col. 4 – col. 5	Col. 6 / (2/4)
1	**2**	**3**	**4**	**5**	**6**	**7**
0	0	-	0	-	0	0
2	8	-	8	0	8	16
4	20	0	20	8	12	24
6	43	8	51	20	31	62
8	80	20	100	51	49	98
10	110	51	161	100	61	122
12	130	100	230	161	69	138
14	146	161	307	230	77	154
16	150	230	380	307	73	146
18	142	307	449	380	69	138
20	130	380	510	449	61	122
22	112	449	561	510	51	102
24	90	510	600	561	39	78
26	70	561	631	600	31	62
28	52	600	652	631	21	42
30	38	631	669	652	17	34
32	27	652	679	669	10	20
34	20	669	689	679	10	20
36	15	679	694	689	5	10
38	10	689	699	694	5	10
40	5	694	699	699	0	(0) 3
42	2	699	701	699	(2)	(4) 0
44	0	699	699	701	(-2)	(-4) 0

5.11. Use and Limitations of Unit Hydrograph :

A. Use :

1. the development of flood hydrograph for extreme rainfall magnitudes for use in the design of hydraulic structures.

2. Extension of flood – flow records based on rainfall records.

3. Development of flood forecasting and warning systems based on rainfall.

B. Limitations :

1. Precipitation must be from rainfall only. Snow – melt runoff cannot be satisfactory represented by unit hydrograph.

2. The catchment should not have unusually large storages in terms of tanks, ponds, large flood banks storages, etc. which affect the linear relationship between storage and dischareg.

3. If the precipitation is decidely nonuniform, unit hydrographs cannot be expected to give good results.

Example (11) : A catchment of 200 hectares area has rainfalls of 7.5 cm, 2 cm and 5 cm in 3 consecutive days. The average Φ index can be assumed to be 2.5 cm/day. Distribution graph percentage of the surface runoff which extended over 6 days for every raainfall of 1 day duration are 5, 15, 40, 25, 10 and 5. Determine the ordinates of the discharge hydrograph by neglecting the base flow.

Solution :

Time Interval (day)	Rainfll (cm)	Infiltration Loss (cm)	ER (cm)	A.D.R %	Distributed Runoff for ER			Runoff	
					5	0	2.5	cm	m^3 / s
1 − 0	7.5	2.5	5	5	0.25			0.25	5.79
2 − 1	2	2.5	0	15	0.75	0		0.75	17.36
3 − 2	5	2.5	2.5	40	2	0	0.125	2.125	49.19
4 − 3				25	1.25	0	0.375	1.625	37.62
5 − 4				10	0.5	0	1	1.5	34.72
6 − 5				5	0.25	0	0.625	0.875	20.25
7 − 6				0	0	0	0.25	0.25	5.79
8 − 7							0.125	0.125	2.89
8 - 9							0	0	0

Runoff of 1 cm in 1 day = (200*100*100)/(86400*100) m^3/s for 1 day = 0.23148 m^3/s for 1 day

Chapter Six

Floods

6.1. Flood: is an unusually high stage in a river, normally the level at which the river overflows its banks and inundates the adjacent area. The hydrograph of extreme floods and stages corresponding to flood peaks provide valuable data for purposes of hydrologic design. Further, of the various characteristics of the flood hydrograph, probably the most important and widely used parameter is the flood peak. At a given location in a stream, flood peaks vary from year to year and their magnitude constitutes a hydrologic series which enable one to assign a frequency to a given flood peak value.

To estimate the magnitude of a flood peak, the following alternative methods are available :

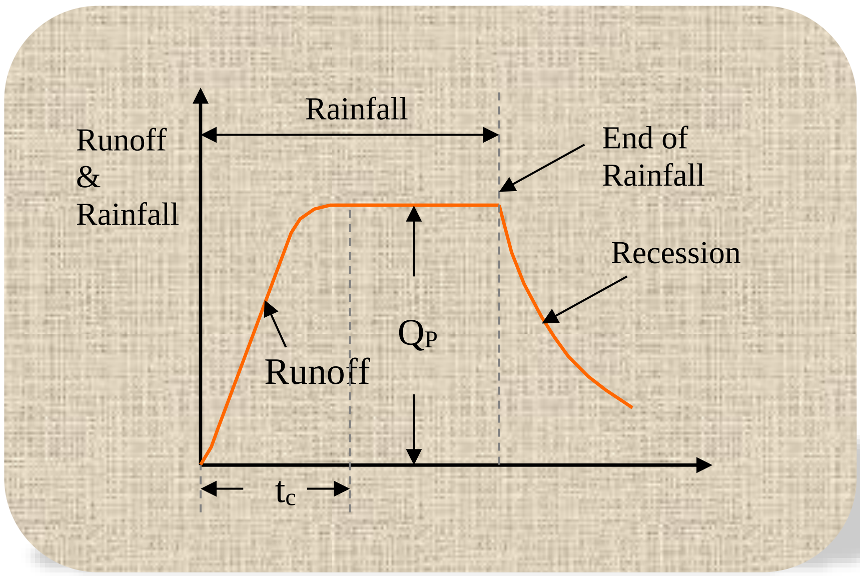
1. Rational Method
2. Empirical Method
3. Unit hydrograph Technique
4. Flood – Frequency Studies

The use of a particular method depends upon :

I. The desired objective
II. The available data
III. The important of the project

Note : the rational formula is only applicable to small size catchments (< 50 km^2) and the unit hydrograph method is normally restricted to moderate size catchments with areas less than 5000 km^2.

6.2. Rational Method : Consider a rainfall of uniform intensity and very long duration occuring over a basin. The runoff rate gradually increases from zero to a constant value as shown in figure below :



The runoff increases as more and more flow from remote areas of the catchment reach the outlet. Designating the time taken for a drop of water from the farthest part of the catchment to reach the outlet as t_c = time of concentration, it is obvious that if the rainfall continues beyond t_c, the runoff will be constant and at the peak value. The peak value of the runoff is given by :

$$Q_P = C\,A\,i \quad \ldots\ldots (1) \qquad t \geq t_c$$

QP : peak discharge (m3/s)

C = Runoff / Rainfall

A : Catchment area (km2)

i : Rainfall intensity (mm/hr)

Equation above is written for field application as :

$$Q_P = \frac{1}{3.6} C\,(i_{tcp})\,A \quad \ldots\ldots (2)$$

itcp : mean rainfall intensity (mm/hr) for a duration equal to (tc) and an exceedance probability (P).

6.3. Time of Concentration : there are many empirical equations to estimate the time of concentration such as :

6.3.1. U.S.A. Practice : if discharge basins of a catchment is small, the concentration time is approximately equal to the time of peak flow :

$$t_c = t_P = C_{tL} \left(\frac{LL_{ca}}{\sqrt{S}}\right)^n \quad \text{..... (3)}$$

t_c : Concentration time (hr)

n = 0.38

s : catchment weighted slope

C_{tL}= constant

L : catchment length and its measured along the water stream from catchment division line (km)

L_{ca} : the distance along the water stream from gauge station to a point on a water stream at the center line of catchment (km)

6.3.2. Kirpich Equation :

$$t_c = 0.01947\, L^{0.77}\ S^{-0.385} \quad \text{..... (4)}$$

tc : concentration time (min)

L : maximum length of travel of water (m)

S = Δ H / L (catchment slope)

Δ H : difference in elevation between the most remote point on the catchment and the outlet.

Example (1) : **An urban catchment has an area of 85 ha. The slope of the catchment is 0.006 and the maximum length of travel of water is 950 m. The maximum depth of rainfall with a 25 year return period is as below :**

Duration (min)	5	10	20	30	40	60
Depth of Rainfall (mm)	17	26	40	50	57	62

If a culvert for drainage at the outlet of this area is to be designed for a return period of 25 years, estimate the requiredd peak flow rate, by assuming the runoff coefficient a 0.3.

Solution:

$t_c = 0.01947 * (950)^{0.77} * (0.006)^{-0.385} = 27.4$ min.

$$\frac{50-10}{10} * 7.4 + 40 = 47.4 \text{ mm}$$

$$i_{tcp} = \frac{47.4}{27.4} * 60 = 103.8 \text{ mm/hr.}$$

$$Q_p = \frac{0.3 * 103.8 * 0.85}{3.6} = 7.35 \text{ m}^3\text{/s.}$$

6.4. Empirical Formulae : **the empirical formulae used for the estimation of the flood peak are essentially regional formulae based on statistical correlation of the observed peak and important catchment properties. To simplify the form of the equation, only a few of the many parameters affecting the flood peak and most of them neglect the flood frequency as a parameter.**

6.4.1. Flood Peak – Area Relationships : **the simplest form of empirical formulae are those which relate the flood peak to the drainage area. The maximum flood discharge Q_p from a catchment area A is given by these formulae as :**

$$Q_P = f(A) \quad ….. (5)$$

1. **Dickens Formula**

$$Q_P = C_D\, A^{3/4} \quad \ldots\ldots (5)$$

Q_P : maximum flood discharge (m^3/s)
A : catchment area (km^2)
C_D : Dickens constant (6 − 30)

2. **Ryves Formula**

$$Q_P = C_R\, A^{2/3} \quad \ldots\ldots (6)$$

Q_P : maximum flood discharge (m^3/s)
A : catchment area (km^2)
C_R : Ryves constant
= 6.8 for areas within 80 km from the coast
= 8.5 for areas within 80 – 160 km from the coast
= 10.2 for limited areas near hills

3. **Inglis Formula**

$$Q_P = \frac{124\,A}{\sqrt{A+10.4}} \quad \ldots\ldots (7)$$

Q_P : maximum flood discharge (m^3/s)
A : catchment area (km^2)

4. **Fuller's Formula :**

$$Q_{TP} = C_f\, A^{0.8}\,(1+0.8 \log T) \quad \ldots\ldots (8)$$

Q_{TP} : (m^3/s) maximum 24- hr flood with a frequency of T- year
C_f : Fuller's constant (0.18 − 1.88)

5. Baird – McIIIWraith Formula

$$Q_{MP} = \frac{3025\ A}{(278+A)^{0.78}} \quad ..... (9)$$

Example (2) : Calculate the maximum flood discharge using an empirical formula and for a catchment area of 40.5 km^2 ?

Solution :

$Q_P = 6 * (40.5)^{0.75} = 96.3\ m^3/s$

$Q_P = 6.8\ (40.5)^{2/3} = 80.2\ m^3/s$

$$Q_P = \frac{124 * 40.5}{\sqrt{40.5 + 10.4}} = 704\ \ m^3/s$$

$$Q_{MP} = \frac{3025 * 40.5}{(278 + 40.5)^{0.78}} = 1367\ m^3/s$$

6.5. Unit Hydrograph

Hydrograph technique can be used to predict the record of hydrograph peak, if the characteristics of the rain causing flooding and infiltration characteristics in addition to the unit hydrograph are available.

6.6. Flood Frequency Studies:

Another approach to the prediction of flood flows, and also applicable to other hydrological processes such as rainfall etc. is the statistical method of frequency analysis.

The values of the annual maximum flood from a given catchment area for large number of successive years constitute a hydrologic data series called the annual series. The data are then arranged in decreasing order of magnitude and the probability P of each event being equalled to or exceeded (plotting position) is calculated by the plotting position formula:

$$P = m / (N+1) \quad \ldots\ldots (10)$$

And

$$T = 1 / P \quad \ldots\ldots (11)$$

Thus, for example, the probability of occurrence of the event r times in n successive years is given by :

$$P_{r,n} = \frac{n!}{(n-r)!r!} P^r q^{n-r} \quad \ldots. (12)$$

Chow (1951) has shown that most frequency distribution functions applicable in hydrologic studies can be expressed by the following equation known as the "general equation of hydrologic frequency analysis" :

$$X_T = \bar{X} + k\,\sigma \quad \ldots\ldots (13)$$

X_T : value of variate X of a random hydrologic series with a return period T

$\bar{X}$: mean of the variate

σ : standard deviation of the variate

k : frequency factor

some of commonly used frequency distribution functions for the prediction of extreme flood value are :

1. Gumbel's Extreme Value Distribution
2. Log – Pearson Type III Distribution
3. Log Normal Distribution

6.6.1. Gumbel's Method:

Another approach to the prediction of flood flows, and also applicable to other

$$X_T = \overline{X} + k\,\sigma_{n-1} \quad \ldots.\ (14)$$

X_T : Maximum flood values for return period T

$$Y_T = -\left[\ln.\ln\frac{T}{T-1}\right] \quad \ldots.\ (15)$$

$$k = \frac{Y_T - \overline{Y_n}}{S_n} \quad \ldots.\ (16)$$

$$\overline{X} = \frac{\sum f.x}{n} \quad \ldots.\ (17)$$

$$\sigma_{n-1} = \sqrt{\frac{\sum(x-\bar{x})^2}{n-1}} \quad \ldots.\ (18)$$

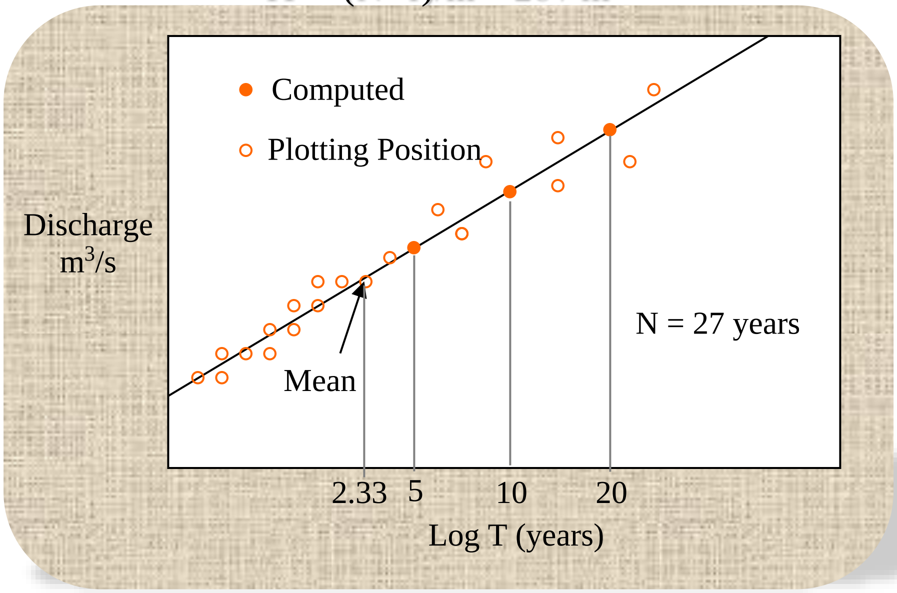
Note: Values of $\overline{Y_n}$ can be obtained from table (7-3) page [257] and Sn from table (7-4) at the same page depending on the n value.

Example (3) : Annual maximum recorded floods in a river for the period 1951 to 1977 is given below. Verify whether the Gumbel extreme value distribution fit the recorded values. Estimate the flood discharge with recurrence interval of (i) 100 years (ii) 150 years by graphical extrapolation.

Year	51	52	53	54	55	56	57	58	59	60	61	62	63	64
Max. flood (m^3/s)	2947	3521	2399	4124	3496	2947	5060	4903	3757	4798	4290	4652	5050	6900
Year	65	66	67	68	69	70	71	72	73	74	75	76	77	
Max. flood (m^3/s)	4366	3380	7826	3320	6599	3700	4175	2988	2709	3873	4593	6761	1971	

Solution : the values are arranged in descending order

$$TP = (N+1)/m = 28 / m$$



m	Flood discharge x (m^3/s)	T (year)
1	7820	28
2	6900	14
3	6761	9.33
4	6599	7
5	5060	5.6
6	5050	4.67
7	4903	4
8	4798	3.5
9	4652	3.11
10	4593	2.8
11	4366	2.55
12	4290	2.33
13	4175	2.15
14	4124	2
15	3873	1.87
16	3757	1.75
17	3700	1.65
18	3521	1.56
19	3496	1.47
20	3380	1.4
21	3320	1.33
22	2988	1.27
23	2947	1.21
24	2747	1.17
25	2709	1.12
26	2399	1.08
27	1971	1.04

T = 5 years : $\bar{X} = 4263$, $\sigma_{n-1} = 1432.6$

$Y_T = -[\ln.\ln(5/4)] = 1.5$, $k = (1.5 - 0.5332)/1.1004 = 0.88$

$X_5 = 4263 + (0.88 * 1432.6) = 5522\ m^3/s.$

T = 10 years : $X_{10} = 6499\ m^3/s.$, $X_{20} = 7436\ m^3/s.$

From figure shown above. It is seen that due to the property of the Gumbel's extreme probability paper, these points lie on a straight line. A straight line is drawn through these points. It is seen that the observed data fit well with the theoretical Gumbel's extreme value distribution.

T = 100 year $\Longrightarrow X_T = 9600\ m^3/s.$

T = 150 year $\Longrightarrow X_T = 10700\ m^3/s.$

And by using equations :

$X_{100} = 9558\ m^3/s.$ & $X_{150} = 10088\ m^3/s.$

Example (4) : Flood frequency computations for a river by using Gumbel's method yieided the following results :

Return Period T (years)	Peak Flood (m^3/s)
50	40809
100	46300

Estimate the flood magnitude in this river with a return period of 500 years.

Solution :

$X_{100} = X + k_{100}\ \sigma_{n-1}$

$X_{50} = \overline{X} + k_{50}\ \sigma_{n-1}$

$(k_{100} - k_{50})\ \sigma_{n-1} = X_{100} - X_{50}$

$= 46300 - 40809 \Longrightarrow (k_{100} - k_{50})\ \sigma_{n-1} = 5491$

$$k_T = \frac{Y_T}{S_n} - \frac{\overline{Y_n}}{S_n}$$

$Y_{100} = -[\ln.\ln(100/99)] = 4.6$, $Y_{50} = 3.9$

$$\left(\frac{Y_{100}-\overline{Y_n}}{S_n}-\frac{Y_{50}-\overline{Y_n}}{S_n}\right)\sigma_{n-1}=5491 \quad \Longrightarrow \quad \sigma_{n-1}/S_n = 5491/(4.6-3.9) = 7864$$

When T = 500 years :

$Y_{500} = -[\ln . \ln(500/499)] = 6.21$

$(Y_{500} - Y_{100}) * (\sigma_{n-1}/S_n) = X_{500} - X_{100}$

$(6.21 - 4.6) * 7864 = X_{500} - 46300$

$X_{500} = 59000\ m^3/s.$

6.7. Confidence Limits:

Since the value of the variate for a given return period, X_T determined by Gumbel's method can have errors due to the limited sample data used, an estimate of the confidence limits of the estimate is desirable. The confidence interval indicates the limits about the calculated value between which the true value can be said to lie with a specific probability based on sampling errors only.

By values x_1 and x_2 given by :

$$X_{1/2} = X_T \pm f(c)\, S_e \quad \text{.... (19)}$$

f(c) : function of the confidence probability and determined from the following table:

C %	50	68	80	90	95	99	95
f(c)	0.674	1	1.282	1.645	1.96	2.58	1.96

S_e = Probable error $= b\dfrac{\sigma_{n-1}}{\sqrt{N}}$ and $b = \sqrt{1+1.3k+1.1k^2}$

$k = \dfrac{Y_T - \overline{Y_n}}{S_n}$ (frequency factor) , N : sample size

Example (5) : Data covering a period of 92 years for a river yielded the mean and standard deviation of the annual flood series as 6437 and 2951 m^3/s respectively. Using Gumbel's method, estimate the flood discharge with a return period of 500 years. What are the (a) 95 % and (b) 80% confidence limits for this estimate.

Solution:

From table (7-3) $\Longrightarrow$ N = 92 , then $Y_n = 0.5589$

From table (7-4) $\Longrightarrow$ N = 92 , then $S_n = 1.202$

$$Y_{500} = -[\ln .\ln (500 / 499)] = 6.21$$

$$K_{500} = (6.21 - 0.5589) / 1.202 = 4.7$$

$$X_{500} = 6437 + 4.7 * 2951 = 20320 \text{ m}^3/\text{s}.$$

$$b = \sqrt{1 + 1.3 * 4.7 + 1.1(4.7^2)} = 5.61 \quad , \quad S_e = 5.61 * \frac{2951}{\sqrt{92}} = 1726$$

A) C = 95 % $\Longrightarrow$ f (c) = 1.96

$$X_{1/2} = 20320 \pm (1.96 * 1726)$$

$X_1 = 23703$ m^3/s , $X_2 = 16937$ m^3/s

B) C = 80 % $\Longrightarrow$ f (c) = 1.282

$$X_{1/2} = 20320 \pm (1.282 * 1726)$$

$X_1 = 22533$ m^3/s , $X_2 = 18110$ m^3/s

6.7. Log – Pearson Type III Distribution:

In this distribution, the variate is first transformed into logarithmic form (base 10) and the transformed data is then analyze. If X is the variate of a random hydrologic series, then the series of Z variate where :

$$Z = \log X \Longrightarrow Z_T = \overline{Z} + k_z \sigma_z$$

k_z : frequency factor which is a function of recurrence interval T and the coefficient of skew (C_s).

σ_z : standard deviationof the z variate sample

$$\sigma_z = \sqrt{\sum (Z-\overline{Z})^2/(N-1)} \quad ..... (20)$$

$$C_s = \frac{N\sum (Z-\overline{Z})^3}{(N-1)(N-2)(\sigma_z)^3} \quad .... (21)$$

$\overline{Z}$: arithmatic mean of z values

N : sample size

Where k_z (C_s , T) from table Page 263

Example (6) : For the annual flood series data given in example 3, estimate the flood discharge for a return period of (a) 100 years (b) 200 years (c) 100 years by using log – Pearson Type III distribution.

Solution:

year	51	52	53	54	55	56	57	58	59	60	61	62	63	64
Max. flood (m^3/s)	2947	3521	2399	4124	3496	2947	5060	4903	3757	4798	4290	4652	5050	6900
Z = log X	3.4694	3.5467	3.38	3.6153	3.5436	3.4694	3.7042	3.6905	3.5748	3.6811	3.6325	3.6676	3.7033	3.8388
year	65	66	67	68	69	70	71	72	73	74	75	76	77	
Max. flood (m^3/s)	4366	3380	7826	3320	6599	3700	4175	2988	2709	3873	4593	6761	1971	
Z = log X	3.6401	3.5289	3.8935	3.5211	3.8195	3.5682	3.6207	3.4754	3.4328	3.588	3.6621	3.83	3.2947	

$\sigma_z = 0.1427$, $\overline{Z} = 3.607$

$$C_s = \frac{27\sum 0.003}{26*25*(0.1427)^3} = 0.043$$

T (year)	K_z	$K_z\ \sigma_z$	Z_T	X_T (m^3/s)
100	2.33	0.3325	3.94	8709
200	2.584	0.369	3.975	9440

Chapter Seven

Flood Routing

7.1. Flood Routing: is a technique of determining the flood hydrograph at a section of a river by utilizing the data of flood flow at one or more upstream sections. The hydrograph logic analysis of problems such as flood forecasting, flood protection, reservoir design and spillwaydesign invariablyinclude flood routing. In these applications two broad categories of routing can be recognised. These are :

1. Reservoir Routing

2. Channel Routing

In reservoir routing, the effect of a flood wave entering a reservoir is studied. Knowing the volume – elevation characteristics of the reservoir and the outflow – elevation relationship for the spillways and other outlet structures in the reservoir, the effect of a flood wave entering the reservoir is studied to predict the variations of reservoir elevation and outflow discharge with time. This form of reservoir routing is essential :

1. **In the design of the capacity of spillways and other reservoir outlet structure.**
2. **In the location and sizing of the capacity of reservoirs to meet specific requirements.**

In channel routing, the change in the shape of a hydrograph as it travels down a channel is studied. By considering a channel reach and an input hydrograph at the upstream end, this form of routing aims to predict the flood hydrograph at various sections of the reach.

7.2. Hydrologic Storage Routing:

Two commonly used semi – graphical methods and a numerical method are described below :

7.2.1 Modified Pul's Method :

$$\left(\frac{I_1+I_2}{2}\right)\Delta t+\left(S_1-\frac{Q_1\,\Delta t}{2}\right)=\left(S_1+\frac{Q_2\,\Delta t}{2}\right)\;....\;(1)$$

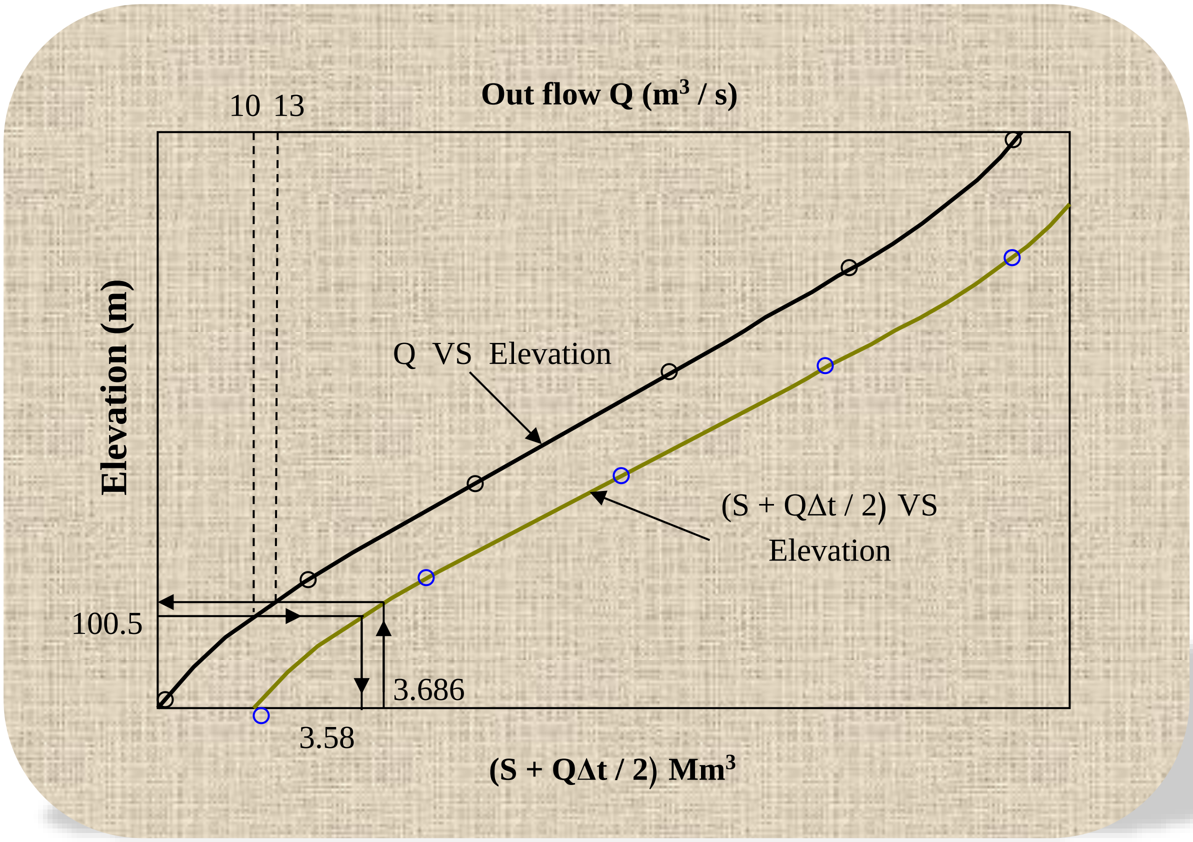
Q_1 , Q_2 : Outflow values at the beginning and end of (Δt)

I_1 , I_2 : Inflow values at the beginning and end of (Δt)

S_1 , S_2 : Storage values at the beginning and end of (Δt)

The following semigraphical method is very convenient for this practice and may be summarized by the following steps :

1. **From the known storage – elevation and discharge elevation data, prepare a curve of $\left(S+\frac{Q\,\Delta t}{2}\right)$ vs elevation (as shown in figure below). Here Δt is any chosen interval, approximately 20 to 40 % of the time of rise of the inflow hydrograph.**



2. **On the same plot, prepare a curve of outflow discharge vs elevation.**
3. **The storage, elevation and outflow discharge at the starting of routing are known. For the first time interval Δt , $\left(\frac{I_1+I_2}{2}\right)\Delta t$ and $\left(S_1+\frac{Q_1\,\Delta t}{2}\right)$ are known and hence by equation (1), the term $\left(S_2+\frac{Q_2\,\Delta t}{2}\right)$ is determined.**
4. **The water – surface elevation corresponding to $\left(S_2+\frac{Q_2\,\Delta t}{2}\right)$ is found by using the plot of step (1). The outflow discharge Q_2 at the end of time step Δt is found from plot of step (2).**
5. **Deductiing ($Q_2\,\Delta t$) from $\left(S_2+\frac{Q_2\,\Delta t}{2}\right)$ gives $\left(S-\frac{Q\,\Delta t}{2}\right)_1$ for the beginning of the next time step.**
6. **The procedure is repeated till the entire inflow hydrograph is routed.**

Example (1) : A reservoir has the following elevation, discharge and storage relationship :

Elevation(m)	Storage (10^6 m^3)	Outflow discharge (m^3/s)
100	3.35	0
100.5	3.472	10
101	3.88	26
101.5	4.383	46
102	4.882	72
102.5	5.37	100
102.75	5.527	116
103	5.856	130

When the reservoir level was at 100.5 m, the following flood hydrograph entered the reservoir :

Time (hr)	0	6	12	18	24	30	36	42	48	54	60	66	72
Q (m^3/s)	10	20	55	80	73	58	46	36	55	20	15	13	11

Route the flood and obtain (i) the outflow hydrograph (ii) the reservoir elevation vs time curve during the passage of the flood wave.

Solution:

A time interval Δt = 6 hr is chosen. From the available data the elevation – discharge – $\left(S+\frac{Q\,\Delta t}{2}\right)$ as shown in the table below :

$\Delta t = 6 * 60 * 60 = 0.0216 * 10^6$ sec.

Elevation (m)	Discharge Q (m^3/s)	(Mm^3)$\left(S+\frac{Q\,\Delta t}{2}\right)$
100	0	3.35
100.5	10	3.58
101	26	4.16
101.5	46	4.88
102	72	5.66
102.5	100	6.45
102.75	116	6.78
103	130	7.26

Then the relationship between Q vs elevation and $\left(S+\frac{Q\,\Delta t}{2}\right)$ vs elevation are plotted as in figure above. At the beginning of routing, the elevation is 100.5 m , Q = 10 m^3/s and $\left(S-\frac{Q\,\Delta t}{2}\right)$ = 3.36 Mm^3 then from this value, Pul's equation can be used to determine $\left(S+\frac{Q\,\Delta t}{2}\right)$ at the end of time step for the first 6 hours :

$$\left(S+\frac{Q\Delta t}{2}\right)_2 = (I_1+I_2)\frac{\Delta t}{2} + \left(S-\frac{Q\,\Delta t}{2}\right)_1$$

= (10+20) * (0.0216 /2) + (3.362) = 3.686

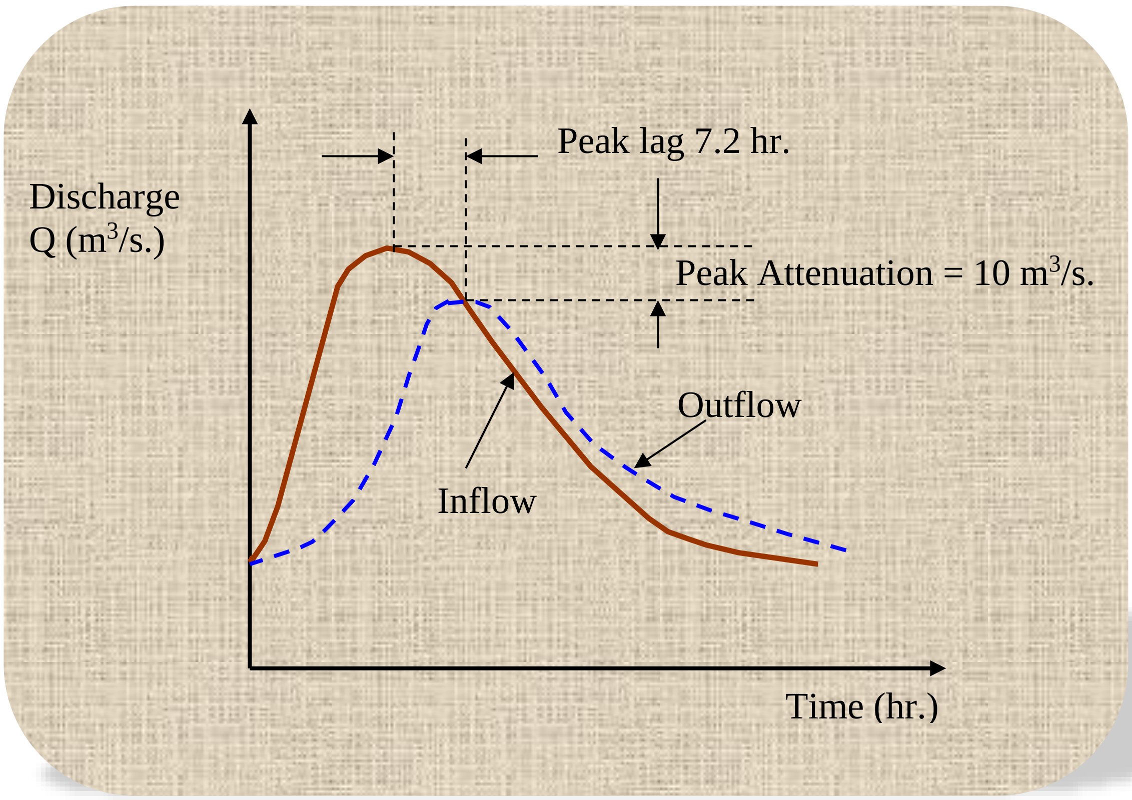
Referring to figure, the elevation of water surface against $\left(S+\frac{Q\,\Delta t}{2}\right)$ = 3.686 Mm^3 is 100.62 m and Q = 13 m^3/s.

For the next time step, the initial value for $\left(S-\frac{Q\,\Delta t}{2}\right)$ = $\left(S+\frac{Q\,\Delta t}{2}\right)$ for the previous step :

3.686 – 13 * 0.0216 = 3.405 Mm^3

The process is repeated for the entire duration of the inflow hydrograph in a tabular form as in the table below :

Time (hr)	Inflow I (m^3/s)	I (m^3/s)	IΔt (Mm^3)	$\left(S - \frac{Q\,\Delta t}{2}\right)$ (Mm^3)	$\left(S + \frac{Q\,\Delta t}{2}\right)$ (Mm^3)	Elevation (m)	Q (m^3/s)
0	10	15	0.324	3.362	3.686	100.5	10
6	20	37.5	0.81	3.405	4.215	100.62	13
12	55	67.5	1.458	3.632	5.09	101.04	27
18	80	76.5	1.652	3.945	5.597	101.64	53
24	73	65.5	1.415	4.107	5.522	101.96	69
30	58	52	1.123	4.096	5.219	101.91	66
36	46	41	0.886	3.988	4.874	101.72	57
42	36	31.75	0.686	3.902	4.588	101.48	45
48	55	37.5	0.513	3.789	4.302	101.3	37
54	20	17.5	0.378	3.676	4.054	100.1	29
60	15	14	0.302	3.557	3.859	100.93	23
66	13	12	0.259	3.47	3.729	100.77	18
72	11			3.427		100.65	14



7.2.2. Goodrich Method :

$$(I_1 + I_2) + \left(\frac{2S_1}{\Delta t} - Q_1\right) = \left(\frac{2S_2}{\Delta t} + Q_2\right) \;....\; (2)$$

Example (2) : Route the following flood hydrograph through the reservoir of previous example by Goodrich method.

Time (hr)	0	6	12	18	24	30	36	42	48	54	60	66
Inflow (m^3/s)	10	30	85	140	125	96	75	60	46	35	25	20

The initial conditions are : when t = 0, the reservoir elevation is 100.6 m.

Solution:

$\Delta t = 6 * 60 * 60 = 0.0216 * 10^6$ sec.

Elevation (m)	Discharge Q (m^3/s)	(Mm^3) $\left(S + \frac{Q\,\Delta t}{2}\right)$
100	0	310.2
100.5	10	331.5
101	26	385.3
101.5	46	451.8
102	72	524
102.5	100	597.2
102.75	116	627.8
103	130	672.2

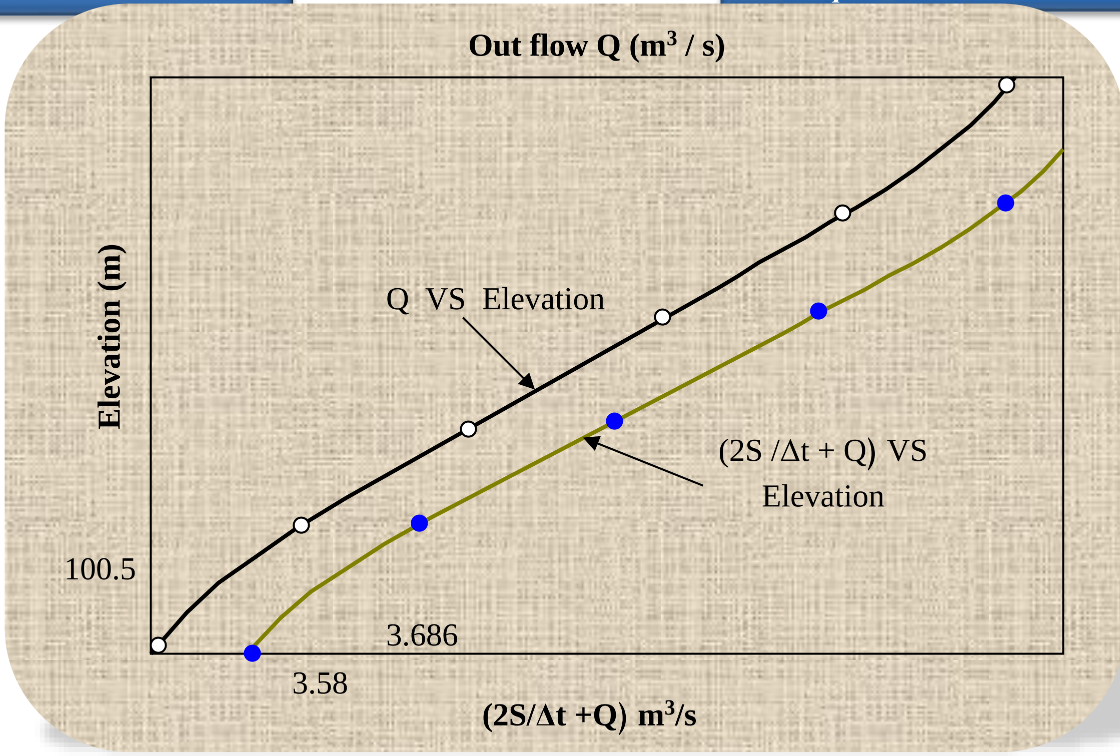
Then the relationship is plotted between :

1. Q vs elevation
2. $\left(S + \frac{Q\,\Delta t}{2}\right)$ vs elevation

When t = 0 , elevation = 100.6 m, from figure :

$Q = 12\ m^3/s \Longrightarrow (2S/\Delta t + Q) = 340\ m^3/s$

$(2S/\Delta t - Q) = 340 - 2 * Q = 340 - 2*12 = 316\ m^3/s$



$(2S/\Delta t + Q)_2 = (10 + 30) + 316 = 356$

For the last value, the elevation = 100.74 m and Q = 17 m³/s

$(2S/\Delta t - Q)_1 = 356 - 2*17 = 322$ m³ / s

Time (hr)	Inflow I (m³/s)	$I_1 + I_2$ (m³/s)	$\left(\frac{2S}{\Delta t} - Q\right)$ (Mm³)	$\left(\frac{2S}{\Delta t} + Q\right)$ (Mm³)	Elevation (m)	Q (m³/s)
0	10	40	316	340	100.6	12
6	30	115	322	356	100.74	17
12	85	225	357	437	101.38	40
18	140	265	392	582	102.5	95
24	125	221	403	657	102.92	127
30	96	171	400	624	102.7	112
36	75	135	391	571	102.32	90
42	60	106	380	526	102.02	73
48	46	81	372	486	101.74	57
54	35	60	361	453	101.51	46
60	25	45	347	421	101.28	37
66	20		335	392	101.02	27

7.3. Hydrologic Channel Routing:

In reservoir routing presented in the previous section, the storage was a unique function of the outflow discharge, S = f(Q). However, in channel routing the storage is a function of both outflow and inflow discharges and hence a different routing method is needed.

7.3.1. Muskingum Equation :

$$Q_2 = C_o I_2 + C_1 I_1 + C_2 Q_1 \quad .... (3)$$

$$C_o = \frac{-kx+0.5\,\Delta t}{k-kx+0.5\,\Delta t} \quad .... (4)$$

$$C_1 = \frac{kx+0.5\,\Delta t}{k-kx+0.5\,\Delta t} \quad .... (5)$$

$$C_2 = \frac{k-kx-0.5\,\Delta t}{k-kx+0.5\,\Delta t} \quad .... (6)$$

where

$C_o + C_1 + C_2 = 1$

k : storage duration constant

x : weighted factor

Example (3) : Route the following flood hydrograph through a river reach for which k = 12 hr and x = 0.2. At the start of the inflow flood, the outflow discharge is 10 m^3/s.

Time (hr)	0	6	12	18	24	30	36	42	48	54
Inflow (m^3/s)	10	20	50	60	55	45	35	27	20	15

Solution:

$I_1 = 10$ $\qquad$ $C_1 I_1 = 4.29$

$I_2 = 20$ $\qquad$ $C_o I_2 = 0.96$

$Q_1 = 10$ $\qquad$ $C_2 Q_1 = 5.23$

$$Q = 10.48\ m^3/s$$

Time (hr)	I (m^3/s)	0.048 I_2	0.429 I_1	0.523 Q_1	Q
0	10	0.96	4.29	5.23	10
6	20	2.4	8.58	5.48	10.48
12	50	2.88	21.45	8.61	16.46
18	60	2.64	25.74	17.23	32.49
24	55	2.16	23.6	23.85	45.61
30	45	1.68	19.3	25.95	49.61
36	35	1.3	15.02	24.55	46.93
42	27	0.96	11.58	21.38	40.87
48	20	0.72	8.58	17.74	33.92
54	15				27.04

For the next time step, 6 to 12 hr, $Q_1 = 10.48\ m^3/s$. The procedure is repeated for the entire duration of the inflow hydrograph. The computations are done in a tabular form as shown in table abov. By plotting the inflow and outflow hydrographs, the attenuation and peak lag are found to be 10 m^3/s and 12 hr respectively.